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Rural Domestic Water System Peak Flows and Design Innovations, Optimal Water Planning Series

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RURAL DOMESTIC WATER SYSTEM PEAK FLOWS AND DESIGN INNOVATIONS

Trevor C. Hughes
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and
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OPTIMAL WATER PLANNING SERIES

PRJER030-3

Utah Water Research Laboratory
College of Engineering
Utah State University
Logan, Utah 84322

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January 1977

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RURAL DOMESTIC WATER SYSTEM PEAK FLOWS AND DESIGN INNOVATIONS

by

Trevor C. Hughes,
Yukio Kono,
and
Ronald Canfield

The work upon which this report is based was supported in part with funds provided by the U.S. Department of the Interior, Office of Water Research and Technology, as authorized under the Water resources Research Act of 1964, Public Law 88-379 (as amended), Project A-030-UTAH, Agreement No. 14-34-0001-7094.

Utah Water Research Laboratory
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PRJER030-1

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ABSTRACT

Planning engineers commonly use generous factors of safety for peak flow estimates in urban water supply systems both as a hedge against unforeseen growth and because economies of scale result in relatively low user costs even with such reserve capacity. Transplanting of such design criteria into the rural setting, however, simply does not work. The low density portions of rural domestic systems require very realistic design criteria or the construction costs become infeasible for the small number of customers involved.

Peak instantaneous flow rates in a Utah rural system were measured continuously during two summers on three dead-end lines serving various numbers of customers. The second summer included measurement of flows to customers whose maximum flow rate was limited by a simple orifice placed in each meter. Conclusions which emerged from this study included: 1) Actual peak demands were lower than those required for design purposes by some state regulatory agencies, but higher than the Farmers Home Administration minimum standard. 2) Where extremely small mains are required by the economics of low density situations, or where unforeseen growth is overtaxing system capacity, peak demands can be cut significantly by simple, flow restricting devices at each meter without decreasing the quality of water service to the customer. 3) Field measurements of head loss through 10 year old plastic pipe indicated a Hazen Williams friction factor average of 133.

ACKNOWLEDGMENTS

This is the final report of a project which was supported in part with funds provided by the Office of Water Research and Technology of the United States Department of the Interior as authorized under the Water Resources Research Act of 1964, Public Law 88-379. The work was accomplished by personnel of the Utah Water Research Laboratory, Utah State University. The authors wish to acknowledge the excellent cooperation and valuable assistance of the Lapoint Culinary Water Company, particularly Mr. Laris Wooley, and various personnel of the Utah State Division of Health and Division of Water Resources.

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INTRODUCTION

NEED FOR ACCURATE ESTIMATES OF DESIGN PARAMETERS

The municipal water supply planning engineer who for the first time is confronted with the task of designing a low density rural domestic water system is likely to discover a surprisingly different array of design problems. The hydraulic network which can represent a very complex component of the municipal design due to the typically large number of loops becomes a very simple, almost deterministic, task in the truly rural setting because of the almost total absence of loops. On the other hand, estimates of peak flows in pipes serving various small numbers of families on rural dead-end lines becomes a very difficult but vitally important task in the rural setting. This problem simply does not exist in the municipal planning problem because fire flow governs such pipe sizes in this range.

The term rural water system as used herein refers to the truly rural low density domestic water system. Such systems may include small community centers in which fire protection and looped mains are completely feasible and this portion of the design may be identical with the municipal design problem; however, the typical rural system also includes very long lengths of pipe which serve two to six families per mile on dead-end lines. Such outlying families may be served by pipe varying from 2 to 4 inch diameters or even 1-½ inches. The focus of this research is on low density portions of the rural system in which conventional fire protection is simply not feasible.

In this context there are three distinct design parameters which at the present state-of-the-art are all judgment type factors. As a result, they cause significant disagreements between consulting engineers, regulatory agencies, and financing agencies. All three factors also are crucial parameters for determining ultimate system capacities, having almost equal importance in determining hydraulic design flowrates. These parameters are: (1) allowance for future growth; (2) peak period demand per connection, and (3) design hydraulic friction factor. The importance of each of these parameters in the rural water problem setting is as follows.

Projection of Future Growth

In the urban setting future changes in land use do not represent a major design problem in regard to sizing water distribution systems. There may be some concern over possible future zoning changes which may effect the size of lots and therefore the number of families per acre; but at least the mains can be sized on the assumption that areas which now have some residential use will eventually change to completely developed areas. Therefore, pipe capacity for some reasonably well defined population density can initially be provided. Here again the fire flow criteria make population density irrelevant except for lines serving several hundred connections.

In the rural setting, however, the potential growth problem is perhaps the most difficult decision facing the planner. The population of rural areas in the U.S. has until recently been either relatively stable or decreasing. During the seventies, however, the traditional rural to urban migration has completely reversed directions. This dramatic change now clearly represents a pattern of net migration away from urban areas. During the sixties all but 5 of 26 of the U.S. non-metropolitan regions lost population through migration. In striking contrast the 1970 to 1975 period resulted in 24 of these same 26 rural regions gaining population (Morrison and Wheeler, 1976).

This dramatic trend adds a new dimension to the rural water planner's problem. Should the proposed water system be designed to handle the anticipated growth in the service area during the period covered by the design life of the distribution system (commonly considered to be 40 years)? This might appear to be desirable; but even if the planner were capable of making such growth projections with reasonable accuracy the economic realities usually make that question merely academic. The practical question is: how can the few sparsely located families currently within the service area possibly finance a system which will later deliver several times the initial demand? The usual answer is that even with substantial federal and state subsidy (often 50 percent of the capital investment) there is no way to make such designs feasible.

The possible courses of action are usually limited to one of the following: (1). The supply conduits and major distribution feeder lines are designed with the largest future growth allowance possible since they represent the future backbone of the system; while laterals serving a few users and the pumps and reservoirs are designed with relatively small growth allowances. The rationale for this concept is that certain small laterals and non-buried components can more easily be replaced or expanded later when the demand materializes (and when other additional customers will provide the necessary revenue). (2) The project is abandoned as being not feasible for the current number of customers.

The determination as to which of these courses is followed depends entirely on the design standards and the general growth related philosophy of the planner or regulatory agency involved. The question of growth allowance will not be addressed in this study. The entire discussion of design criteria which follows assumes that the objective is to determine as closely as possible, the short term demand per customer, and that as a separate item, some allowance for growth has been included in the design number of customers. Whether or not that growth allowance is the correct one is not an issue here. The point is that liberal criteria for flow per family with the implicit assumption that this will provide an additional "cushion" for unexpected growth is not a desirable concept in the rural setting. The allowance for growth should be as liberal as possible but it should not be hidden as part of the unit demand.

Peak Period Demand Per Connection

The focus of this study is on the second of the parameters characterized as judgment factors, the short term peak period unit demand (flow rate per connection) used to size distribution system pipes. The reasons that this parameter is still very much subject to disagreement include the following:

Very little reliable empirical data are available. Most rural systems record monthly (or longer) demands at individual meters and many have master meters which record flows from sources into equalizing reservoirs at intervals ranging from one day to one week. However, these data are of little use in predicting the maximum 5 minute demand, for instance, in the system's main line. Further, even if the master meter were located below the equalizing reservoir (as it is in systems which buy from a wholesaler) defining the instantaneous demand requires a special study wherein the meter is either converted to a continuous recorder or is read at intervals of a few minutes during peak hours during many peak days. Either method is expensive and such data are almost never obtained. Even if short term demand rates were known for total systems (based upon master meter readings) this information is of little use in defining design standards for small dead-end laterals. The rural demand function is highly nonlinear in relation to number of families served, particularly when the number of families is small. A constant such as 1.5 gallons per minute per connection (gpmc) may be perfectly adequate for a line serving 500 or 5000 families because the function tends to approach a constant in this range; however 3.0 gpmc capacity may be required for a line serving only 10 families (5.0 if the Utah standard is used). Data which verify short term demands in the range from 4 to 100 services are almost nonexistent.

Other system capacity parameters such as daily (24 hour) and monthly peak demand are, of course, important in sizing source related facilities such as pumps, treatment plants, transmission conduits, and reservoirs. The scope of this report, however, is limited to sizing distribution mains, and therefore to instantaneous flowrates. The other design parameters are addressed in a separate report (Hughes and Israelsen, 1977).

Hydraulic Friction Coefficient

The increasing use of new types of hydraulically smooth pipe such as asbestos cement and plastic and the addition of smooth internal linings to steel and cast iron pipe during the last 20 years has intensified the lack of agreement on what constitutes good design friction coefficients. Since peak flows normally imply turbulent flow, most designers have adopted the empirical Hazen Williams equation rather than the theoretically derived but more cumbersome Darcy equation. As an example of the designer's dilemma, pipe manufacturers recommend a Hazen Williams "C" coefficient of 150 for plastic and 140 for asbestos cement pipe. Current standards of the Utah Division of Health, however, have been increased recently from the traditional C of 100 for metallic pipes to 110 for asbestos cement and 120 for plastic (PVC). There seems to be little question that the newer pipes will deliver the high flows estimated by the high coefficients claimed by the manufacturers for new pipe under laboratory conditions but little is known about the stability of these capacities in the field as the pipe ages. Since flow rate is proportional to this coefficient, a pipe designed for a C of 150 but for which the true C is only 100 (after aging, sand accumulation, rough glued joints, etc.) will deliver only 67 percent of the intended flow rate. Friction losses need further study, particularly in rural system where most of the pipe materials used are now plastic and asbestos cement.

Measurement of friction losses was not a principal objective of this study. However, the required demand flow rate instrumentation did provide an opportunity at little additional cost to make some field measurements of friction losses through long lengths of 10 year old small diameter PVC pipes.

PROJECT OBJECTIVES

The research was addressed to answering three specific questions:

1. What is the probability of instantaneous water demand exceeding any given magnitude (gallons per minute per connection) for rural distribution mains serving small numbers of connections.

2. What is the impact on both quality of service and on peak demand levels of design innovations such as flow restricting devices.

3. What are reasonable friction coefficients for hydraulic design of plastic pipe now being used in rural water systems.

PREVIOUS RESEARCH

There is a large body of literature addressed to various water supply demand parameters. Most of it is focused on demand in urban systems and much of the urban literature is concerned with monthly or daily demands. The primary objective of this study, however, is directed to instantaneous demands and therefore the literature review will be limited to those publications which include information on very short term demands.

In order to provide a comparison between urban and rural domestic water demands the first study discussed will be concerned with urban systems. The remainder will be concerned with rural demands.

Howe and Linaweaver (1967)

The landmark U.S. effort in the area of water demand data collection was the Residential Water Use Research Program of Johns Hopkins University (Howe and Linaweaver, 1967, and Linaweaver et al., 1966). Accumulated flows were recorded on punch tapes at 15 minute intervals at master meters in 41 residential areas over three years. The resulting demand functions indicate that the major factors were economic level of users, climate, and water rates. Climate was found to affect outside use dramatically but had little impact on domestic demand. Rates also had little impact on domestic use but considerable impact on sprinkling use. Actual price elasticity (percent change in price) figures were approximately 22 percent for domestic demand and 70 percent in the West and 157 percent in the east for sprinkling demand (Howe and Linaweaver, 1967). Peak short term demands measured (averages for the 41 systems) were 1.2 gpmc in the eastern U.S. and 1.7 gpmc in the west. Typically several hundred families were served by these lines.

The results of this study and others which are included in the literature review are summarized in Table 1.

Kansas—Williams (undated)

This unpublished study reports the measurement of peak demands in four different low density rural water systems in Kansas. Peak periods were determined by preliminary monitoring of master meters to identify peak months, days of week, and hours of the day. The master meters were then read at 1 minute and 1 hour intervals during several peak period hours and days during 1966 and 1968. The number of connections served varied from 16 to 185. The users are reported as typical for Kansas rural districts. No small towns were included. Demand components include household, livestock, and a minor amount of irrigation. The livestock demand appears to be minor except for a few fairly large dairies which use the system for equipment washing.

The data presented allow computation of peak instantaneous, peak day and month water requirements. The data are summarized in Table 1. The instantaneous peak demands reported are extremely low relative to measurements of other systems. Water rate schedules for these systems were not reported, however, a personal contact with the manager of one of the systems indicated that average water costs were $1.00 per thousand gallons.

Table 1. Peak instantaneous and peak day demands.

Water System	Average Monthly Demand 1000 gal/Conn.	Number of Residences		Highest Measured Instantaneous Peak (gpm/Conn.)	Highest Measured 24 Hour Peak (gpm/Conn.)	Date of Measurements
		Total System	Lateral Measurement			
Oklahoma District #3	7.0	100	37	1.85	0.40	1974
Kansas (Montgomery #6)	4.5	185	185	.32	.165	1968
Kansas (Montgomery #3)	7.4	21	21	.52	.30	1966
Kansas (Montgomery #1)	5.8	36	36		.25	1966
Kansas (Allen #6)		16	16	.75		1968
Kansas (Johnson Summary)		100	100	.90		
Urban Systems -- Special Study						
United States West (Johns Hopkins)	13.7	Averages 44 to 410		1.7 (mean)	.68	1963-1965
United States East (Johns Hopkins)	9.3	Averages 44 to 410		1.2 (mean)	.54	1963-1965

Oklahoma—Goodwin (1975)

This M.S. Thesis reports the measurement of peak demands during 1974 in three different laterals of Rural Water District No. 3, Payne County, Oklahoma. Each lateral served between 34 and 39 users. The three master meters are all located below the system reservoir so that instantaneous demands were obtained (as well as peak day and peak month). Average monthly flows were not reported but can be rather closely estimted from the February through September data which are reported.

The results are summarized in Table 1. The water rates were not reported in the cited publication but personal contact with the water district manager revealed that in 1974 they averaged between $1.50 and $2.00 per 1000 gallons.

The flows were measured with good accuracy and recorded on paper tape in a manner that data for both short term flow rates and accumulative volumes of flow are identifiable. The water users of this system were classified into six categories which consisted of five types of homes (determined primarily by size and value) plus dairies. An interesting conclusion was that no significant difference in demand existed between most sizes of homes. The final grouping of data differentiated only between dairies, the very lowest valued type of residence (only very minimum plumbing) and all others.

Mississippi—Ginn et al. (1966)

The objective of this study was to develop frequency distribution information for instantaneous peaks in a rural water system in Mississippi. Preliminary measurements revealed extremely low demands in the rural system (which are not reported). The project was then modified and subsequent measurements were taken within an urban subdivision. Standards of living in rural Mississippi are expected to increase over time and therefore rural demands were hypothesized to change and eventually to approach those in the urban setting.

The approach used to measure demands was to observe individual meters until peak hours were established and then to read 15 individual meters at 1 minute intervals during the 2 to 3 hour morning and evening peaks. These measurements were aggregated to create a typical residence demand distribution and a statistical model was then used to predict the probability of any combination of daily peak events occurring simultaneously at any desired number of identical residences. The resulting design criteria will be discussed later and compared with the Utah study results.

Livestock Demand (Iowa)—Schultz and Austin (1976)

This study emphasizes the difference between demand patterns of rural and urban systems which

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Livestock Demand (Iowa)—Schultz and Austin (1976)

This study emphasizes the difference between demand patterns of rural and urban systems which may result when a rural system includes large livestock operations. Instantaneous demands were recorded continuously on two laterals serving 10 and 30 farms each on the Hospers Rural Water System No. 1 in northwestern Iowa. Weekly meter readings of individual farms were also recorded and detailed survey forms indicating the number and weight of livestock at each farm were obtained. Demand functions were determined for 11 different types of livestock as follows: feeder cattle, stock cows, calves, hogs, cows, small pigs, lactating dairy cattle, non-lactating dairy cattle, dairy calves, poultry, and turkeys.

This study is an excellent reference for livestock demands in terms of gallons per day; however, despite the fact that flows were monitored continuously, instantaneous flow rates are not reported. Peak periods of the day due to the superimposed domestic and livestock demands are discussed and compared to residential only peak hours but flow rates are deleted. Domestic demands are included in the predictive equations as 40 gpd per person.

Johnson (1968)

The author of this paper is an engineer with the Farmers Home Administration in Kansas (a state where the FmHA water system program has been very active). This paper articulates the general FmHA philosophy toward rural water design criteria and describes the Kansas experience with minimum standard innovations. It distinguishes between suburban systems which "...have a way of evolving into urban systems," and truly rural systems. The objective of suburban system design is described as providing a skeleton for the urban system which will follow. The truly rural system, however, is characterized as facing at most a modest growth and more likely a loss of population.

The experience of the FmHA water supply program since 1937 is characterized as having an excellent loan repayment record and experiencing dramatic growth and acceptance by rural groups and consulting engineers. The design standards are described as follows.

> Not being a research agency, we do not have the equipment or personnel needed to make extensive studies. We have, however, through the years, been forced to make enough measurements to establish safe design limits. Measurements on existing systems in Kansas show that simultaneous peaks for systems with 100 taps average 0.90 gpm per tap, exclusive of fire flows. This figure varies inversely with the number of taps though not necessarily as a straight line. Most Kansas engineers are presently using for a design a minimum simultaneous peak flow of 2 gpm, adjusted for estimated expansion potential. This has proven adequate in all cases; in some it may be extravagant.
>
> Some engineers are using an approach which allows higher individual flows at the far end of the system. This is usually done by allowing a larger figure, such as 10 gpm to the terminal tap, and 100 gpm for 100 taps. These two values are plotted on a semilog scale and a straight line drawn between them. Flows for intermediate taps are taken from the line. By this method, 30 taps would be allowed 76 gpm and 60 taps 90 gpm. This method seems to come close to the actual demand pattern.
>
> A second method which comes closer to the patterns we have observed in the field is the insertion of a third point, 40 gpm for ten taps, and connecting the three points.
>
> These minimum demands may seem low. No doubt they are by municipal standards. This can be accounted for, at least in part, by the fact that despite all possible economies, this is expensive water. The average minimum charge for water on rural systems in Kansas is about $7.00. The average cost per thousand for 20,000 gallons, about $1.00. This tends to reduce waste. It also discourages lawn and garden irrigation.

These design criteria are used on systems averaging customer density of two per mile of pipe. Wide use of plastic pipe has made possible this low density without unreasonable user costs for the water delivered. In addition to the systems described above which deliver peak demands, an alternative concept is being used in Kansas and Colorado in areas which may have only one customer per mile. This is the constant flow system. These systems depend upon individual storage cisterns at each service into which a very small flow (approximately 1 pint per minute) is delivered. The water is repressurized by a small pump on demand. This concept allows extremely small pipe diameters over long distances and therefore much lower costs. A comparison of the peak demand vs. contant flow designs for a particular Kansas system suggest a construction cost saving of 75 percent by using the latter and estimated user rates of $20 and $6.50 per month respectively.

Because of pressure variations within such systems the unmetered constant flows to each customer are regulated by inexpensive compression type flow restrictors which were developed specifically for this use.

DEMAND MEASUREMENT PROCEDURE AND SCOPE

DESCRIPTION OF INSTRUMENTATION

Lapoint Culinary Water Incorporated, a rural system in northeastern Utah, was selected for the pilot study. It includes several long small diameter dead-end lines (which minimized the number of master meters required). The system was constructed ten years ago using PVC pipe for all small lines and appears to have essentially no leakage in the lines selected for the study (separating leakage from customer use was therefore not required). The capacity of the small diameter lines in the sample area had been questioned by the State Division of Health even before construction of the system and considerable growth has occurred since construction; therefore a current study of pipeline flows and minimum pressures was of considerable practical value to the water utility officers in assessing the limits of their distribution system capacity.

Flowrate Measurements

The approach used to assess peak demands was to install orifice type meters at three locations: A 2-½" diameter line serving 22 families (25 during 1976); a 2" line serving 12 families (15 during 1976) and 1-½" line serving 4 families. The master meter and service locations are shown in Figure 1. A flow meter recorder (differential pressure transducer) with a 24 hour ink chart was used to convert the orifice pressures to flowrates and to record them continuously (Figure 2). The three orifices were installed on June 26, 1975, and flows were recorded continuously until the possibility of frost damage to the recorder required stopping the operation on October 23, 1975. The recorder was operated again during the summer of 1976 (July 11 to September 26). The summer data are considered to represent the peak season on this system. Budget limitations allowed purchase of only one recorder, so it was rotated at approximately 2-week intervals among the 3-meter stations in order to obtain an intensive record at each station under varying climatic conditions. During the second summer three additional services (all mobile homes) were added to the system so that the number of families metered was increased as noted above.

The orifices and flow meter were designed and manufactured by Honeywell. The calibration of each unit was checked at Utah Water Research Laboratory prior to installation in the field and found to be accurate to two significant figures (to the near gpm for the range of flows encountered). The flow recorder indicated instantaneous flow rates but did not integrate them. The chart drive consisted of a manual wind 7-day clock so that no electrical power was required at the site.

Pressure Measurements

In addition to flowrate, line pressures were also recorded at each flow meter location. Specifically, daily minimum pressures were determined. The devices used were 4" Marshalltown conventional pressure gages to which a second indicator had been added that was pushed by the pressure indicator and which would remain in the minimum position. These were read and reset daily at each of the meter stations during the first year of operation. The purpose of gathering this data was two-fold: (1) In determining demands delivered through water mains the question arises—does the flow rate recorded represent the true demand of the users or is the desired flow during peak periods being limited by hydraulic capacity of the system. If the latter is true, line pressures should become extremely low during peak flow periods and minimum pressure should therefore be a reliable indicator of this condition. (2) Minimum line pressures were expected to be somewhat inversely correlated with maximum flow rates. Depending upon the strength of the correlation, in a particular location, maximum flow might be calculated with reasonable accuracy from minimum pressure measurements. Since it is much cheaper to measure pressure than flow this appeared to be a possible method of extending the flow data base.

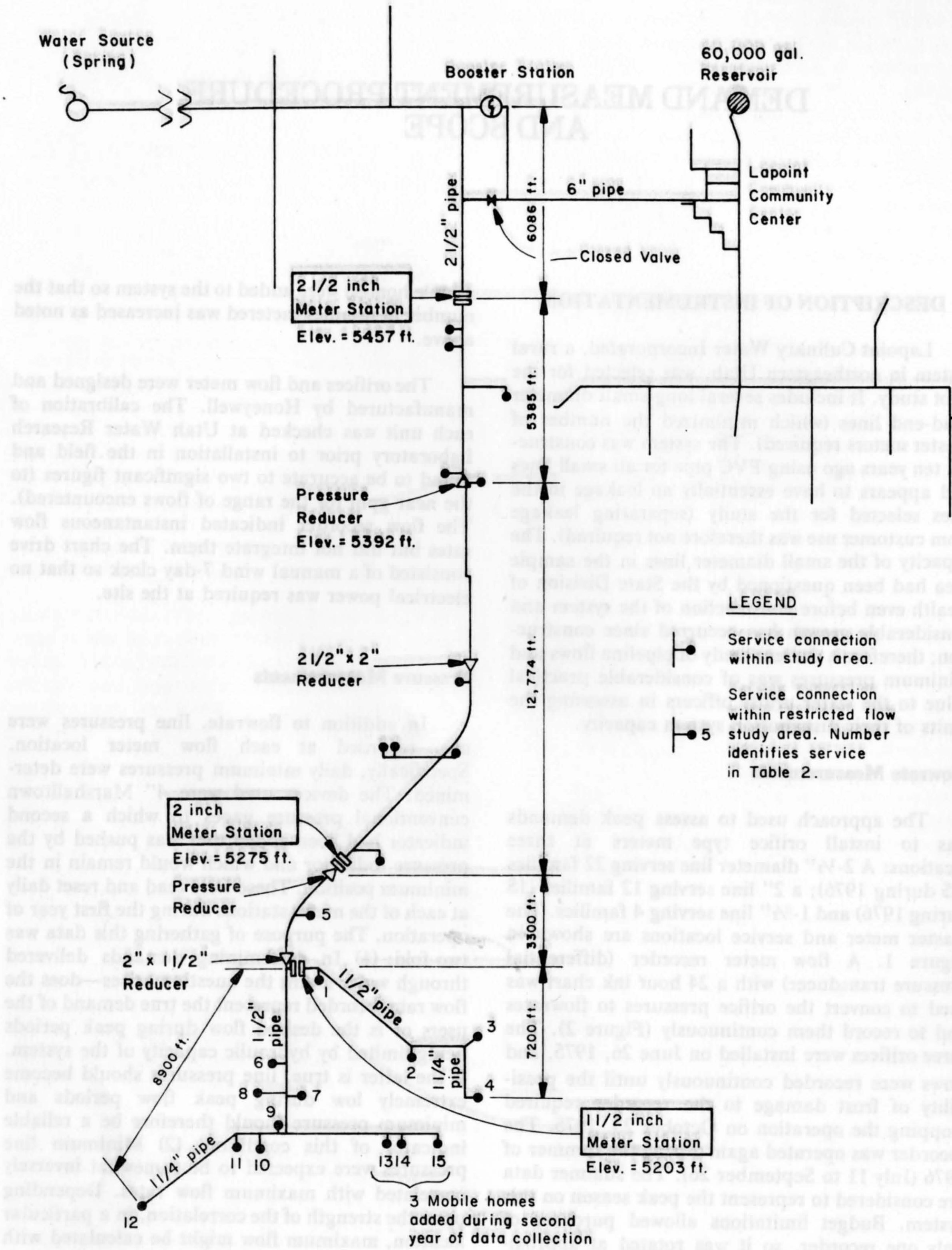

Figure 1. Lapoint water system study area.

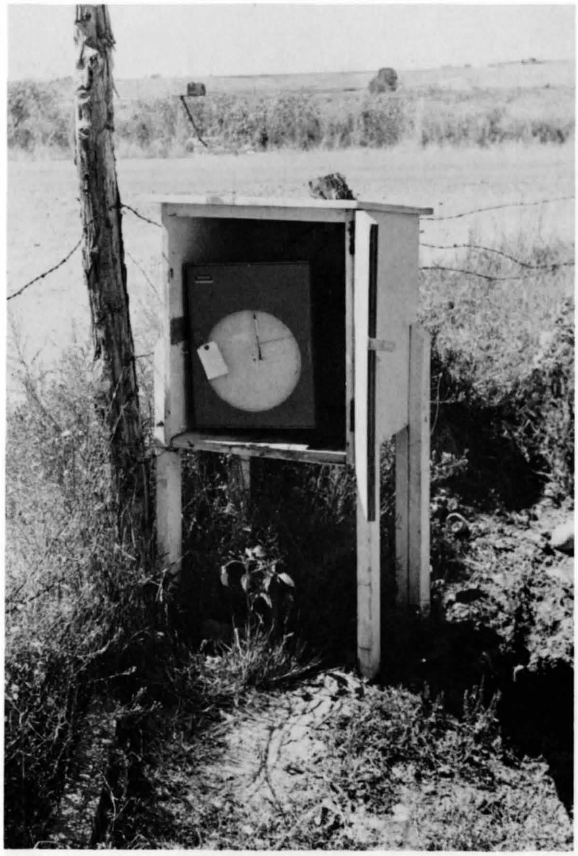

Figure 2. Flowrate recorder field set up.

For example, in this study only one flow recorder was available, but minimum pressures were recorded daily at all three stations. It was hoped that by rotating the flow meter among the three stations, correlations could be developed between flow and pressure from which daily maximum flow rates at each station could be calculated.

Flow Restricting Devices

One of the objectives of this study was to analyze the impact of design innovations such as flow restricting devices on both quality of service to the user and on peak flow rates. The specific devices selected for use during the second year of this study were very thin gage stainless steel orifices with an 0.25" diameter hole drilled at the center. These were installed in the inlet connection of each individual meter on the 12 and the 4 meter connection lines during second summer of operation. The devices were installed between two rubber washers in the ¾" meter inlets. They required absolutely no modification in plumbing and only about 2 minutes each for installation and only pennies per meter. The ¼" diameter orifice produced the desired decrease in maximum possible flow rate at each service within reasonable limits.

DESCRIPTION OF WATER DEMAND IN STUDY AREA

Although the study area included 22 residential services (25 during the second year), only 15 were within the section where flow restrictors were installed. A detailed description of these 15 services is given in Table 2. The remaining 10 customers were not interviewed and therefore the type of detailed information contained in Table 2 is not available for them, however, an inspection of these other residences and their apparent extent of outside water use suggests that Table 2 can be considered to be representative of the entire study area.

The connections served by the metered lines are typically modest farm houses although 7 of 25 have been constructed during the last 5 years. The majority have only one bathroom although 27 percent have two or three bathrooms. All have automatic clothes washers and almost half have automatic dishwashers. The average number of people served is 4.5 per family. Two of the original 22 services were mobile homes (one of which had the largest demand in the study area) but three more were added during the second year of the study. Examples of the types of houses are shown in Figure 3.

The outdoor water demand includes both irrigation of gardens and landscaping and stockwater. However, as is detailed in Table 2, both of these demands are relatively minor. Of 15 customers interviewed one had a major irrigation demand, six irrigated small lawn or garden areas, and four used this system only to supplement water from a canal system.

Although more than half of the families interviewed had some livestock, only three used the domestic system to provide stockwater during the summer period and all three of these were small family barnyard type operations (no beef cattle feedlots).

The Lapoint system reports an average monthly demand of 17,000 gallons per customer. The water costs $10 per month minimum plus $0.20 per 1000 gallons in excess of 6000. This policy results in average unit costs of $0.72 per 1000 gallons.

Table 2. Lapoint service descriptions in orifice test area.

Conn.	Maximum Flow		Quality of Service (I)			Type of Demand							
	original (gpm)	With orifice (gpm)	(a) General adequacy	(b) Bad periods?	(c) Change during orifice use?	(a) Livestock General	(b) What seasons?	Outside (c) irrigation	(d) No. of Bathrooms	(e) Auto. Dishwasher	(f) Auto. Clothes Washer	(g) How many persons?	Remarks
1	---	5.5	not available for interview			Few dairy & chickens	All year	None	--	--	--	2	Uses well inside - this system for stock only
2	Box flooded	6.7	Good	Too much pressure	No	Few dairy cattle	Winter only	Little (supplement ditch only)	1	Yes	Yes	7	
3	10.4	6.1	Good	No	No	None	--	Lawn - front yard	2	Yes	Yes	5	
4	10.3	6.1	Good	No	No	200 cattle	Winter only	Some (supplement ditch)	3	No	Yes	10	
5	18.0	7.0*	Good	Summer of '76 only	yes	Few chickens	All year	Lawn & Garden (8100 + 4000 sf)	1	Yes	Yes	2	Major demand is irrigation
6	12.6	6.7*.	Good	No	No	None	—	Small lawn	1	No	Yes	4	*These orifice capacities not measured but estimated by comparison with similar measured services
7	11.0	6.3*	Good	No	No	Few cattle	Winter only	Lawn - 10 supplement to ditch	1	No	Yes	2	
8	11.6	6.5*	Good	No	No	None	--	No lawn but small garden	1	No	Yes	4	
9	12.8	6.2	Good	No	Yes	Few cattle	Winter only	Small lawn & Very small garden	1	No	Yes	4	
10	5.6	No orifice	not available for interview			None	--	No lawn but very small garden	1	--	--	4	Apparently some restriction in line, therefore, orifice not used
11	5.2-5.4	"	Good	During 1976 only	Yes	None	—	None	2		--	7	(same as 10)
12	12.0	"	Good	No	No	Few cattle & pigs	Pigs only in summer	Lawn - supplement ditch	1	Yes	Yes	6	Note large unrestricted flow despite end of line location
13-15	—	Installed but not tested	(3 Mobile homes - new this summer)		N/A	—	--	None	—	—	--	—	New services in summer '76 Therefore, orifice effect N/A
Summary	11.0 average	6.3 measured average	100% Good	8 No 2 Yes, but 1976 only	7 No 3 Yes	1 large cattle demand but winter only 6 family barnyard type misc. demand but only 3 in summer 5 no livestock in 1975 8 no livestock in 1976		1 large lawn & garden 3 Front lawn only 3 small gardens 4 supplement ditch 5 none (1975) 5 none (1976)	8 with 1	44	100	4.5 Average	

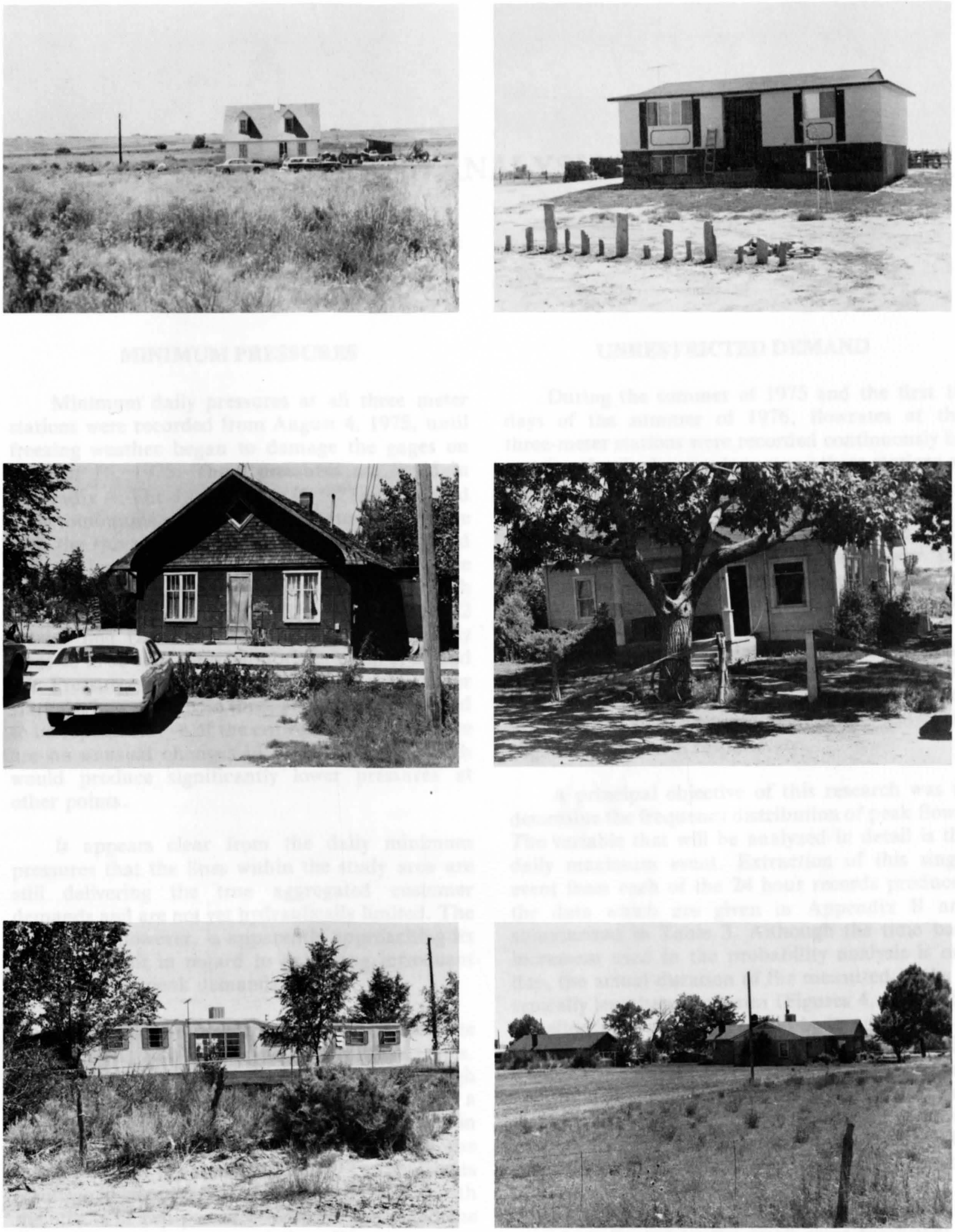

Figure 3. Typical residences in study area.

RESULTS AND ANALYSIS OF DATA

MINIMUM PRESSURES

Minimum daily pressures at all three meter stations were recorded from August 4, 1975, until freezing weather began to damage the gages on October 16, 1975. These pressures are listed in Appendix A. The 4 service line (1-½") experienced daily minimums generally in the 50 to 60 psi range with the three month minimum being 29 psi and the next lowest event at 42 psi. The 12 service line (2") minimums range generally from 70 to 90 with the minimum events being 40 to 46 psi. The 22 service line (2-½") minimums ranged generally from 40 to 60 with three-month minimum of 25 and 30. Pressures were recorded only at the meter stations; however, these three points are considered to be representative of the entire study area. There are no unusual changes in pipeline slopes which would produce significantly lower pressures at other points.

It appears clear from the daily minimum pressures that the lines within the study area are still delivering the true aggregated customer demands and are not yet hydraulically limited. The 2-½" line, however, is apparently approaching its hydraulic limit in regard to delivering infrequent instantaneous peak demands.

The minimum pressure data did not correlate at all well with the peak daily flow rates. Apparently the system's booster pump, although downstream from the study area, still had a significant impact on flow and therefore pressure in the long 6" transmission line serving both the pump inlet and the study area lines. The transients resulting from operation of this pump coupled with the effects of two pressure reducers prevented the desired flow/pressure correlation. No attempt was made therefore to extend the flow rate base by computing flow as a function of minimum pressure. This did not represent a significant problem, however, since the actual measured flow data appeared to represent a completely adequate sample of summer peak demand at all three locations.

UNRESTRICTED DEMAND

During the summer of 1975 and the first 10 days of the summer of 1976, flowrates at the three-meter stations were recorded continuously by rotating the single recorder among these stations at approximately 2-week intervals.

The daily hydrographs recorded for the small number of services involved do not show the pronounced morning and afternoon peaks that are typical of urban system hydrographs. The relatively large impact on total flow of peak demands by a few services produces an almost random distribution of events during the day and almost no flow occurs at night. Typical 24 hour hydrographs for the 4, 12, and 22 service meters are shown in Figures 4, 5, and 6 respectively.

A principal objective of this research was to determine the frequency distribution of peak flows. The variable that will be analyzed in detail is the daily maximum event. Extraction of this single event from each of the 24 hour records produced the data which are given in Appendix B and summarized in Table 3. Although the time base increment used in the probability analysis is one day, the actual duration of the measured peaks is typically less than 3 minutes (Figures 4, 5, and 6). An alternative approach would be to analyze all of the events above a certain flowrate using, for example, 5 minutes as the time base. This would provide a much greater number of data points and therefore smaller confidence limits for a given probability level; however, the more conservative daily maximum events were selected for the analysis.

As indicated by the small skew coefficient in Table 3, the daily peaks are normally distributed. The data were therefore linearized by the normal probability plots shown in Figure 7. Some recorder difficulties caused by high humidity in the below grade vault at the 2" meter station resulted in less usable data (17 days) here than at the other two stations during the first year of operation. It was

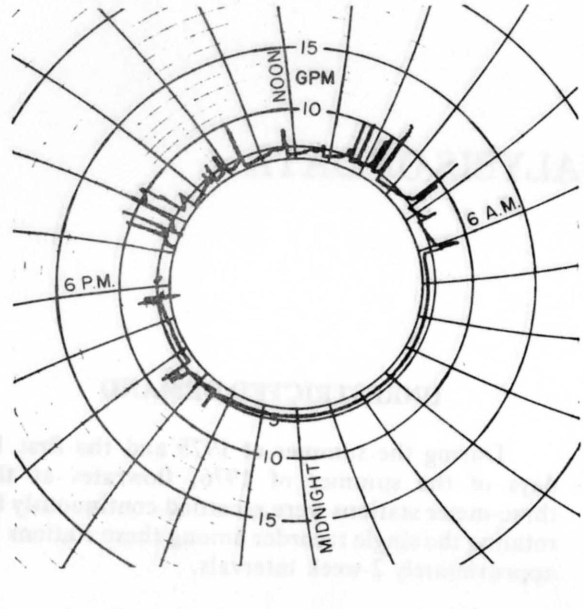

Figure 4. Typical 24 hour hydrograph for 4 service meter.

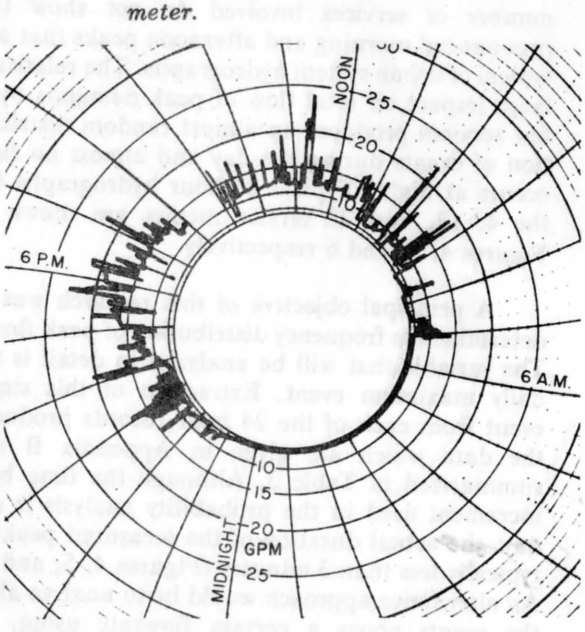

Figure 5. Typical 24 hour hydrograph for 12 service meter.

believed that this smaller sample may account for the relatively flatter slope of the 12 service line; however, an additional 18 days of unrestricted flow data at this station were obtained during 1976. The results were almost identical to those based on the 1975 data. Because of the three additional services on this line during 1976, the combination of data from both years presents difficulties. Since the 1976 data provided no significant changes, the 1975 data only (representing 12 services) will be analyzed in the following discussion. The 1976 data, however, is included in Appendix B. One is

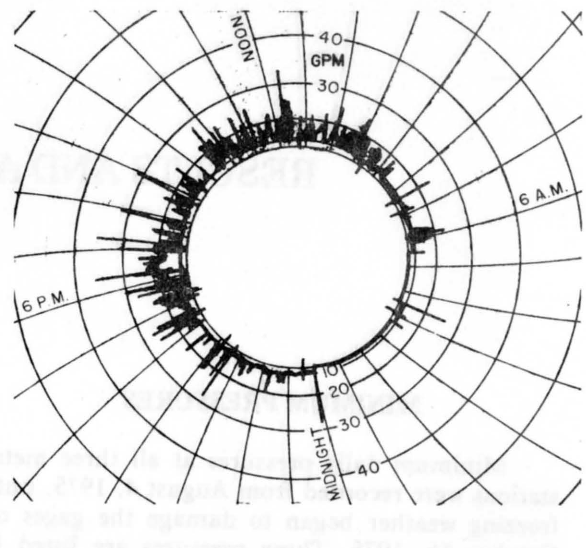

Figure 6. Typical 24 hour hydrograph for 22 service meter.

tempted to show a best fit graphical location of this line which crosses the 22 connection line. However, an analytical approach using the more conservative t distribution (which incorporates the relative reliability due to differences in number of data points) supports the approximate relationship shown in Figure 7.

The probability distribution shown in Figure 7 represents the expected value of unit demand for a given probability level P or recurrence interval t_r (where $P = 1/t_r$) assuming that the computed means and standard deviations of the data equal the true parameters for these normally distributed populations. Actually there is no assurance that these point estimates are precisely equal to the true parameter and so the more conservative t distribution which accounts for randomness in both the means and standard deviations was also determined as follows:

The probability is computed at any risk level (α) that the normalized flow variable exceeds the associated value of the t distribution (Kempthorne and Folks, 1971):

$$P\left(\frac{X-\bar{X}}{S\sqrt{\frac{N+1}{N}}} > t_{1-\alpha,N-1}\right) = \alpha$$

The related value of flow (X) is:

$$X > t_{(1-\alpha,N-1)}\, S\sqrt{\frac{N+1}{N}} + \bar{X}$$

The results of this analysis and a comparison with the graphical solutions from Figure 7 are given

Table 3. Statistical parameters for daily maximum instantaneous flows in gallons per minute (gpm) per service.

Parameter	Number of Services		
	4	12	22
Number of days (N)	30	17	42
Mean daily maximum ($\overline{X}$)	3.17	1.97	1.73
Standard deviation (S)	0.373	0.199	0.2735
Skew coefficient (g)	+ .01901	- .00706	+ .00805
Maximum measured flow	4.0	2.25	2.29

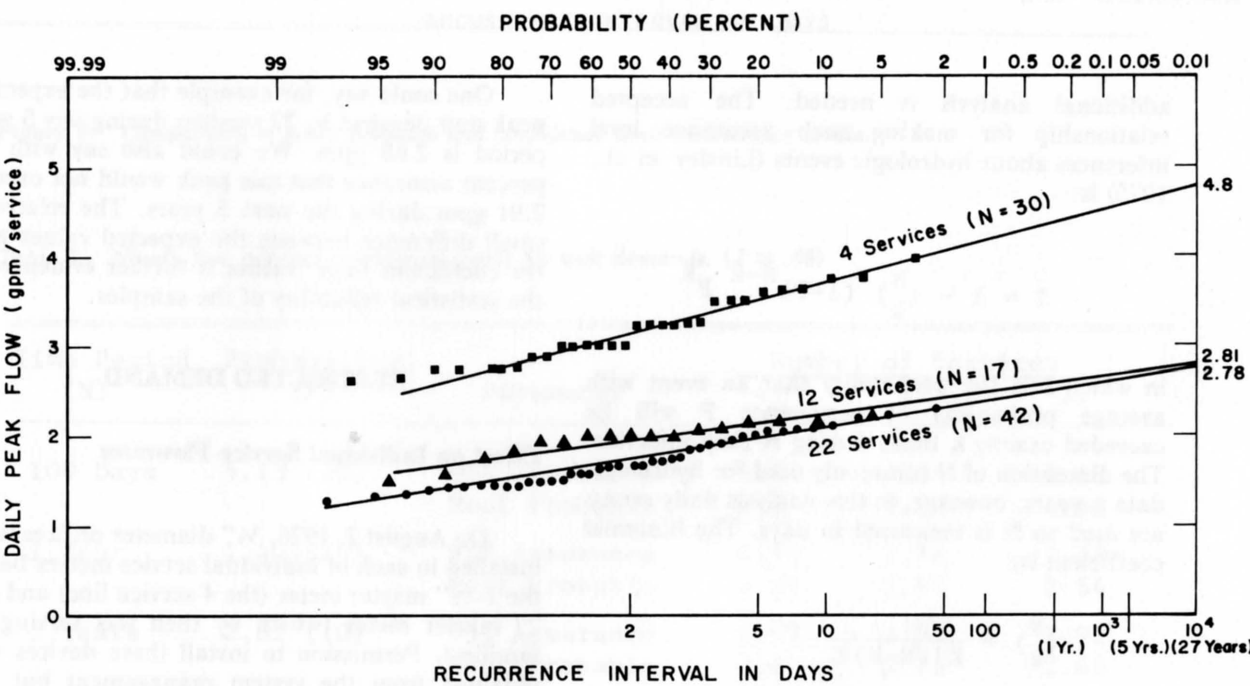

Figure 7. Probability distribution of daily maximum flow data.

in Table 4. The excellent agreement between the t distribution and the graphical solution indicates that the point estimates of the statistical moments computed from the data sample are very close to the true population parameters.

The parameters described previously, however, still represent only the expected values of peak demands during any time interval. In order to estimate confidence limits on flows not being exceeded during a particular time interval, an

Table 4. Comparison of unit demands (gpm/conn) from t distribution and from graphical linearized normal distribution.

Recurrence Interval	Parameter	Number of Services 4	12	22
P = .25	t	.683	.689	.681
t_r = 4 days	$X_{(t)}$	3.43	2.11	1.91
	$X_{(Fig.2)}$	3.38	2.07	1.90
P = .01	t	2.462	2.567	2.421
t_r = 100	$X_{(t)}$	4.10	2.49	2.39
	$X_{(Fig.2)}$	4.18	2.49	2.40
P = .0005	t	3.659	3.965	3.547
t_r = 5.5 yrs.	$X_{(t)}$	4.55	2.78	2.70
	$X_{(Fig.2)}$	4.60	2.71	2.66

additional analysis is needed. The accepted relationship for making such assurance level inferences about hydrologic events (Linsley et al., 1975) is:

$$J = 1 - \binom{N}{k} (1-P)^{N-k}\, P^k$$

in which J is the probability that an event with average probability of occurrence P will be exceeded exactly k times during N time intervals. The dimension of N commonly used for hydrologic data is years, however, in this analysis daily events are used so N is measured in days. The binomial coefficient is:

$$\binom{N}{k} = \frac{N!}{K!(N-k)!}$$

For the special case where k = 0, that is, the largest event,

$$J = 1 - (1-P)^N$$

Figure 8 shows a comparison of expected values of demand levels and 95 percent confidence limits on these levels over the probable range of interest for design standards. This comparison is also given for a few particular values in Table 5.

One could say, for example that the expected peak unit demand by 22 services during any 5 year period is 2.68 gpm. We could also say with 95 percent assurance that this peak would not exceed 2.91 gpm during the next 5 years. The relatively small difference between the expected values and the coefficient limit values is further evidence of the statistical reliability of the samples.

RESTRICTED DEMAND

Effect on Individual Service Flowrates

On August 2, 1976, ¼" diameter orifices were installed in each of individual service meters below the 1-½" master meter (the 4 service line) and the 2" master meter (which by then was serving 15 families). Permission to install these devices was obtained from the system management but the individual families involved were purposely not informed of the flow restriction in their service. It was felt that a much more unbiased opinion of the impact of these devices on the quality individual service could be obtained if the individuals involved were not informed of the experiment until after the data gathering and interviews were completed. It was necessary to shut off water to each family for only a few seconds during installation of the device. When the installation was completed each resident was asked to turn on all possible indoor and outdoor plumbing fixtures for 1 minute while the

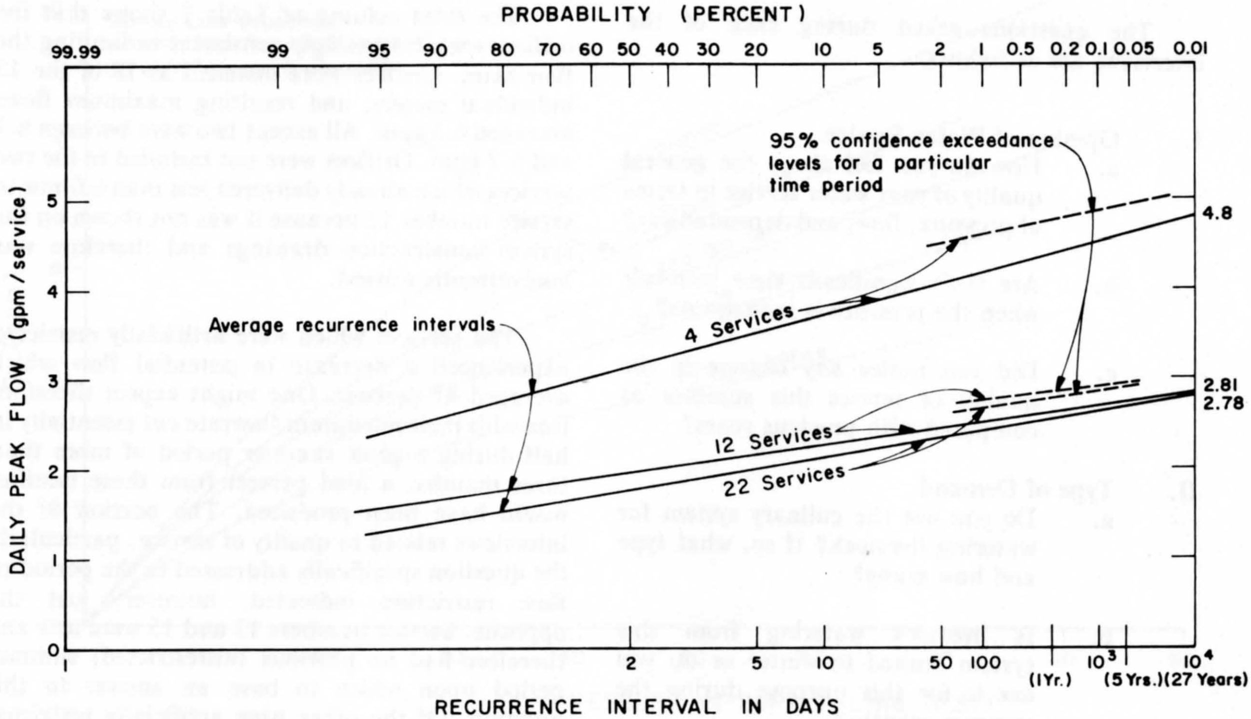

Figure 8. Comparison of most probable and confidence limit exceedance levels.

Table 5. Ninety-five percent confidence levels for unit demands. ($J = .05$)

Time Period (N)	Probability (P)	Parameter	Number of Services 4	12	22
100 Days	$5.13\ (10)^{-4}$	95% Assurance	4.6	2.74	2.69
		Most Probable	4.2	2.52	2.42
1 Year	$1.405(10)^{-4}$	95% Assurance	4.75	2.81	2.78
		Most Probable	4.39	2.62	2.54
5 Years	$2.81\ (10)^{-5}$	95% Assurance	4.93	2.93	2.91
		Most Probable	4.6	2.73	2.68

rate of flow was observed at the meter. Residents were informed only that their water system was being studied in connection with a research project.

The flowrates at master meters on the 4 and 15 service lines were then recorded alternately during the next 2 months. On September 26 cold weather during the evenings presented potential damage to the instrumentation and the recorder operation was terminated.

The orifices were removed on November 19 (after 3-½ months) and interviews with family heads were conducted. The procedure followed was to remove each orifice before contacting the families and then asking them to again turn on the water at all possible fixtures in order to measure the potential maximum unrestricted flowrates. Interviews were then conducted, still without mentioning to any residents the orifice installation or removal.

The questions asked during each of the interviews are as follows:

I. Opinion of Water Service
   a. How do you feel about the general quality of your water service in terms of pressure, flow, and dependability?

   b. Are their significant time intervals when the pressure is inadequate?

   c. Did you notice any change in the quality of service this summer as compared with previous years?

II. Type of Demand
   a. Do you use the culinary system for watering livestock? If so, what type and how many?

   b. Is livestock watering from this system limited to winter or do you use it for this purpose during the summer months?

   c. How much outside irrigation is provided by the culinary system?

   d. How many bathrooms do you have?

   e. Do you have an automatic dishwasher?

   f. Do you have an automatic clothes washer?

   g. How many people live in the house during the summer?

The survey results are summarized in Table 2 and have already been partially discussed in connection with the description of the nature of water demand components. The discussion in this section will be limited to the extent of physical restrictions to potential individual service flowrates and the importance of this restriction to the water users.

As indicated in Table 2, the average of the maximum unrestricted flowrates was 11.0 gpm. Most were between 10.3 and 12.6. One service with a large irrigation nozzle close to the main was able to draw 18 gpm while two others could draw only 5.2 and 5.6 gpm. The latter two services apparently were limited by some characteristic of their own service downstream from their meters because the main line (although only 1-½" diameter at these points) delivered 12 gpm to a service below them.

The third column of Table 2 shows that the orifices were surprisingly consistent in limiting the flow rates. Orifices were installed at 12 of the 15 individual meters, and resulting maximum flows averaged 6.3 gpm. All except two were between 6.1 and 6.7 gpm. Orifices were not installed in the two services which already delivered less than 6.0 nor in service number 12 because it was not shown on the system construction drawings and therefore was inadvertently missed.

The services which were artificially restricted experienced a decrease in potential flow which averaged 47 percent. One might expect therefore that with their maximum flowrate cut essentially in half during a peak summer period of more than three months, a loud protest from these families would have been produced. The portion of the interviews related to quality of service, particularly the question specifically addressed to the period of flow restriction indicated, however, just the opposite. Service numbers 13 and 15 were new and therefore had no previous (unrestricted) summer period upon which to base an answer to this question. Of the other nine artificially restricted services, only two had observed any change during the period of orifice installation. One of these, as might be expected, was the service with the large irrigation system which had its maximum flow reduced by 61 percent. The other family which noticed a change was the one closest to the three new connections and they attributed the change (perhaps correctly) to the additional demand of these new services. These results seem to indicate that maximum potential flow rates from ¾ services are called upon so rarely that reducing them by as much as 50 percent has no significant impact upon the quality of service. This should not be a surprise when one considers that in order to produce the maximum potential service flow rate three or four fixtures have to be operated fully open simultaneously. The probability of this happening during normal residential water service operation approaches zero. After installation of the flow restricting device, water users were apparently able to operate two or three fixtures simultaneously (representing normal peak use) with little observable decrease in the capacity of those fixtures.

The impact on maximum flow rate of various size orifices as a function of main line pressure is displayed in Figure 9. These functions were computed from basic hydraulic relationships for flow in pipe, minor losses, and the following orifice head loss equation:

$$Q = KA\sqrt{2gh}$$

in which Q is flowrate, K is a orifice coefficient which varies with relative diameters of the orifice

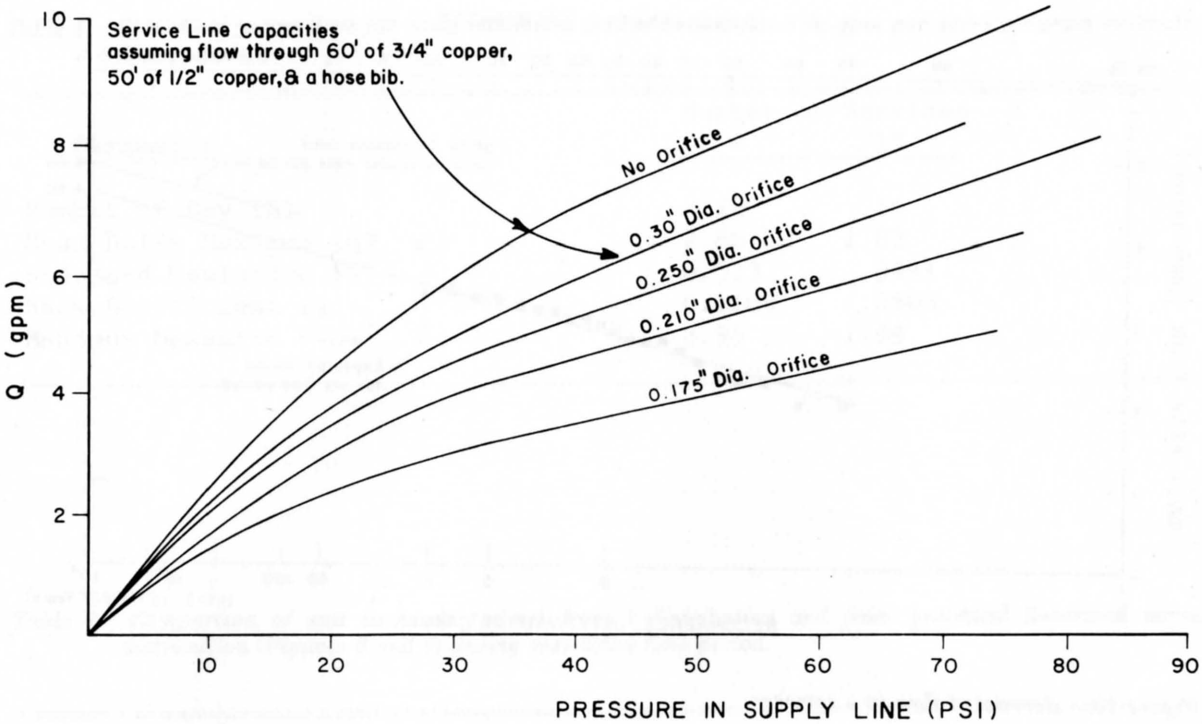

Figure 9. Influence of various orifice sizes on service capacity.

and pipe, A is area of the orifice, g is the acceleration of gravity, and h is head loss at the orifice (some of which is recoverable). Empirical measurements of flow through a ¾" water service similar to that described in Figure 9 indicated that for the service and orifice size used in this study 0.72 is an approximate value for K. As shown in the figure, systems with 50 to 60 psi main line pressure and a 0.25" diameter orifice at each meter would be expected to deliver 6 to 7 gpm per service. This matches very closely the maximum restricted flows delivered by the Lapoint system. This implies that the service pipe lengths and sizes represented by Figure 9 are a reasonably accurate hydraulic model of the average of the services in the Lapoint study area.

As indicated by Figure 9, the service capacity varies from 5.2 to 7 gpm as the line pressure varies from 40 to 65. This seems to be a reasonable variation for the objectives of this study. Commercial devices are available which produce a flatter flow/pressure function by means of a compressible diaphragm which restricts flow more at higher pressures than at lower. These devices, however, have a laying length of more than 2 inches and therefore would require a considerable installation cost (excavation, plumbing, and back fill), as compared to the orifice which can be completely installed or removed in approximately 2 minutes. Since the hydraulic characteristics of the orifice produce a reasonably flat curve at high flows it was preferred to the more expensive and more permanent commercial devices.

Effect on Distribution System Flowrates

The daily maximum flowrates at the 4 and 15 service master meters are given in Appendix C. These data were analyzed both analytically and graphically in the same manner as the unrestricted flows. The graphical linearized probability functions are shown in Figures 10 and 11. The usual statistical parameters and the t distribution analyses are shown in Tables 6 and 7. Table 7 also includes values of peak flow from the linearized frequency analysis for selected recurrence intervals. Table 8 compares these expected values for any recurrence interval with the 95 percent confidence limits for a particular time period.

Table 6 shows a surprisingly large decrease in standard deviation between the 4 and 15 service demands. This trend was also present in the unrestricted flow data (Table 3) but not nearly so pronounced. This decrease in variability is also reflected in the differences between expected and 95 percent confidence levels in Figures 10 and 11.

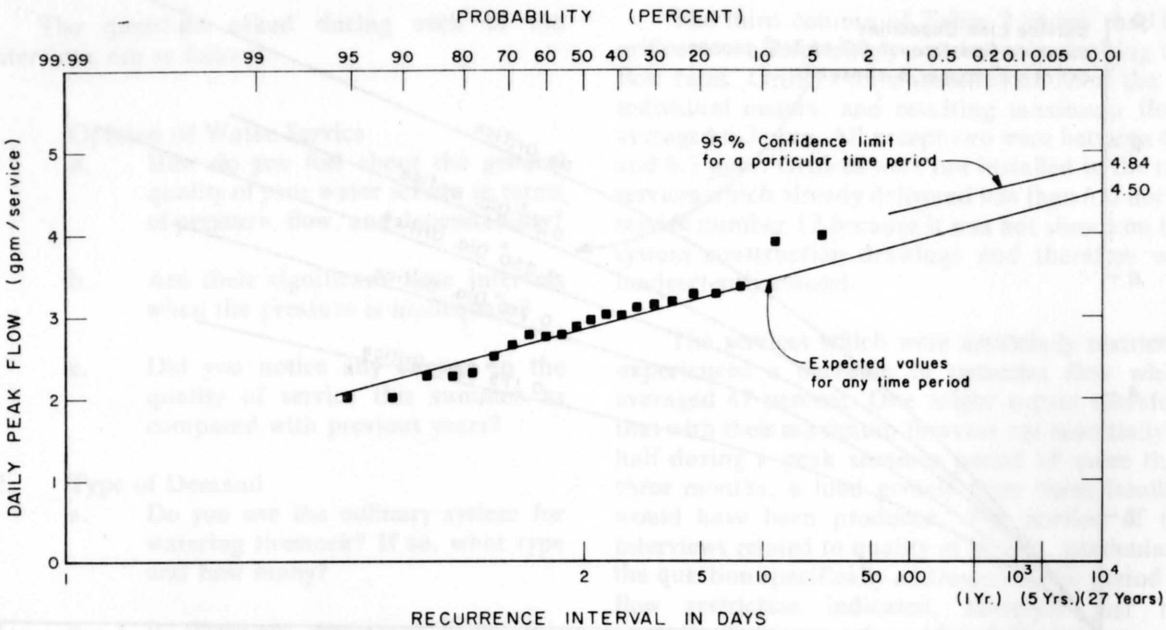

Figure 10. Restricted flow to 4 services.

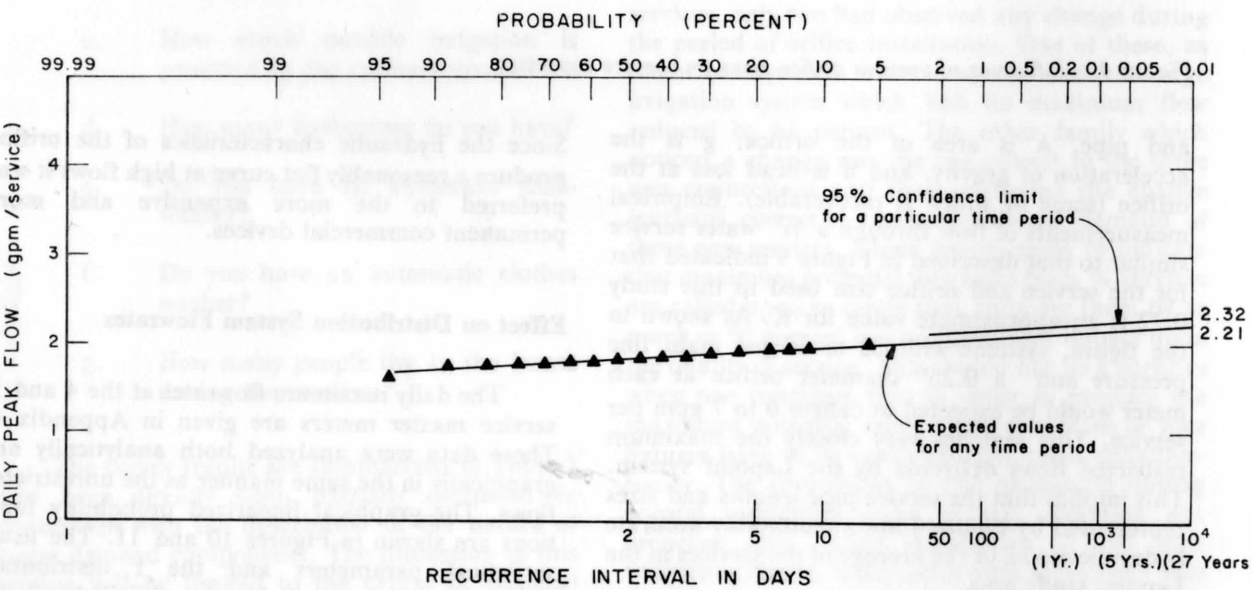

Figure 11. Restricted flow to 15 services.

The large variation in the 4 service data is partly due to two unusually high flow events which are displayed in Figure 10. The two highest measured flows do not fit the linearized function nearly so well as in the unrestricted case. These two events were essentially ignored in fitting the data in Figure 10, but not in the analytical analysis; hence the much greater difference between the t distribution flow and the graphical flow for the 4 service demand at long recurrence interval (see Table 7, t_r = 5.5 years). The 15 service meter data were linearized very well, however, and agreed closely with the t distribution analysis as before.

A logical conclusion seems to be that the smaller the number of services involved, the less impact flow restricting devices have upon infrequent instantaneous flows in distribution lines. On the other hand, the mean of the daily peak was reduced by 10 percent at the 4 service meter and only 7 percent at the 15 service meter by the orifices.

Table 6. Statistical parameters for daily maximum instantaneous flows in gpm per service during restricted flow periods.

Parameter	Number of Services 4	15
Number of Day (N)	22	16
Mean Daily Maximum ($\bar{q}$)	2.86	1.83
Standard Deviation (S)	0.523	.0981
Skew Coefficient (g)	0.1109	.0503
Maximum Measured Flow	3.95	1.99

Table 7. Comparison of unit demands (gpmc) from t distribution and from graphical linearized normal distribution (Figures 8 and 9) during restricted flow period.

Recurrence Interval	Parameter	Number of Services 4	15
$p = .25$	t	.686	.691
$t_r = 4$ days	q(t)	3.23	1.90
	q(fig.)	3.13	1.90
$p = .01$	t	2.518	2.60
$t_r = 100$ days	q(t)	4.21	2.09
	q(fig.)	3.88	2.08
$p = .0005$	t	3.82	4.07
$t_r = 5.5$ years	q(t)	4.9	2.24
	q(fig.)	4.3	2.18

Table 8. Ninety-five percent confidence levels per unit demands during restricted flow period. (J=.05).

Time Period (N)	Probability (P)	Parameter	Number of Services 4	15
100 days	$5.13(10^{-4})$	95% Assurance	4.3	2.175
		Most Probable	3.88	2.08
1 year	$1.405(10^{-4})$	95% Assurance	4.46	2.21
		Most Probable	4.10	2.11
5 years	$2.81(10^{-5})$	95% Assurance	4.64	2.26
		Most Probable	4.3	2.18

The 15 service demand in general, experienced a substantial decrease in peak flowrates during the flow restriction period. Table 9 compares peak flows with and without the orifices at selected recurrence intervals at both master meters. The decrease in 4 service demand for infrequent events was approximately 7 percent while the 15 service demand decrease approached 20 percent. While it is true that increasing the number of services from 12 to 15 during the study would be expected to decrease unit demand at this meter, the comparison between the 12 and 22 service demands suggests that this effect would be negligible.

PEAK FLOW DURATION ANALYSIS

As shown in the daily hydrographs (Figures 4, 5, and 6) the measured maximum flowrate events were of very short duration. This is of interest in considering criteria for design standards. The selection of design flowrates should logically involve decisions on both recurrence interval and duration of design flow events. For example, if one designs for an event which is expected to occur say once in 5 years, how should that event be defined? Should it be the maximum level of flow which lasts for 1 minute, 10 minutes, or 30 minutes?

In order to analyze the duration of peak flow events, the three highest flow days (unrestricted) at each master meter station were analyzed. The length of time that the flowrate was above a given level during each of these days is given in Appendix B. The averages of the duration function for the three peak days at each location are shown in Figure 12. The 4 and 22 service demands consistently shown a marked decrease in flowrate between 1 and 5 minute events; while the 12 service demand was much flatter. The reason for this difference is unknown but it undoubtedly explains the lack of difference between the 12 and 22 service demands at long recurrence intervals. (See Fgure 7.) For example, if 5 minute duration events, rather than absolute daily peaks, had been used in developing the frequency analysis, Figure 12 suggests that there would be a much greater difference between the 12 and 22 service demand functions in Figure 7.

Table 10 displays demands lasting various time intervals per day as a percent of daily peak events. The quantities in Figure 12 and Table 10 do not represent durations of single events but rather the total daily time (for peak days) during which flow equalled or exceeded a particular flowrate.

FRICTION COEFFICIENT MEASUREMENTS

The Lapoint water system was constructed during 1966. During the summer of 1967 measurements of head loss through two sections of small diameter PVC lines at various flowrates were recorded by Hughes. One of these sections of the distribution system which was approximately 2 miles long was within the study area of this report. The other section was approximately 1 mile in length. Both sections of line delivered water at head losses that would have been predicted by the Hazen Williams equation using a friction coefficient (C) averaging 151. The range of variation in calculated C values for individual measurements was 141 to 158.

Table 9. Impact of flow restricting orifices on daily peak flow rates.

	4 Services			12 to 15 Services		
Recurrence Interval	Without Orifices	With Orifices	Percent Decrease	Without Orifices	With Orifices	Percent Decrease
Average	3.17	2.86	10	1.97	1.83	7
4 days	3.38	3.13	7	2.07	1.90	8
100 days	4.18	3.88	7	2.49	2.08	16
1 year	4.39	4.10	7	2.62	2.11	19
5 years	4.6	4.30	7	2.73	2.18	20
27 years	4.8	4.50	6	2.81	2.21	21

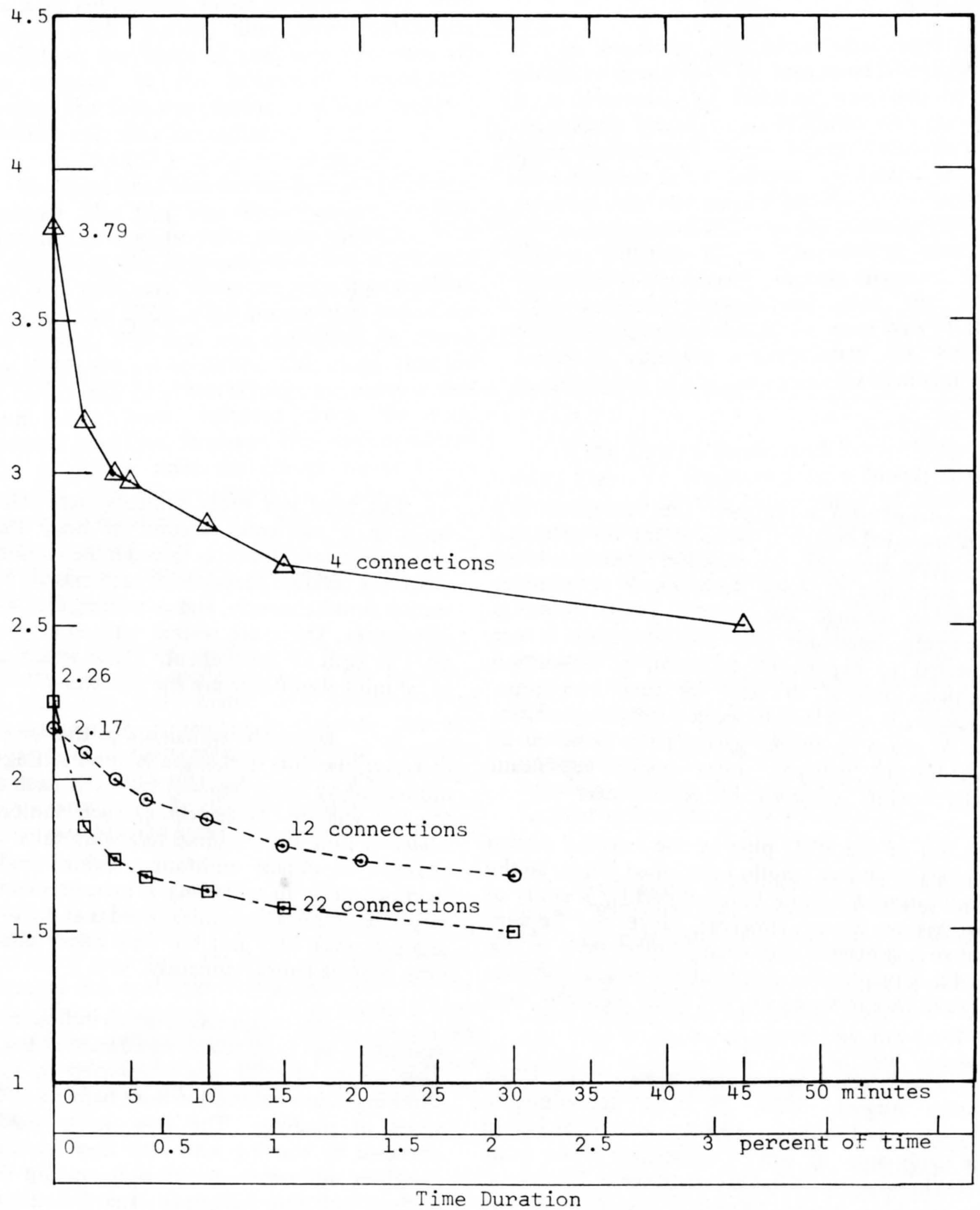

Figure 12. Averages of the top three highest time duration curves.

Table 10. Flowrates lasting various durations and percent of daily maximums ( ).

Number of Services	(gmp) Daily Maximum	Time 3	Duration 5	(Minutes) 15	30
4	3.79	3.18 (84)	3.1 (82)	2.71 (72)	2.6
12	2.17	2.04 (94)	1.98 (91)	1.8 (83)	1.68 (77)
22	2.26	1.8 (80)	1.71 (76)	1.59 (70)	1.5 (66)

This tended to support the manufacturer's recommended design C value of 150 for extruded PVC pipe. However, the question remained—does this hydraulic capacity (smoothness or effective diameter) change over time? Plastic pipe is extremely chemically inert and therefore is not expected to experience corrosion or deposition problems typical of steel or cast iron pipe. However, is sand deposition a potential problem; and do glued vs. rubber gasket joints or 40 vs. 20 foot lengths between joints justify significant differences in design friction coefficients?

All of the PVC pipe in the Lapoint system consists of 40 foot lengths with glued joints, so the latter question cannot be answered by a study of this system alone. However, since the Lapoint system had been in operation 10 years prior to this study, it provided a potential for producing data related to variations in hydraulic capacity over time.

The friction loss measurements were taken during August, 1976. In order to eliminate transients and unmeasured flows at services within the study area, all service connection meter stops were closed during these measurements. Because of the resulting inconvenience to customers it was necessary to limit the duration of this study to a few hours at each of the three sections of the distribution system that were involved. It was therefore not possible to obtain a large number of replications of the measurements in order to improve accuracy. The measurements that were obtained, however, appear to be reasonably consistent with the exception of one obvious error.

The head loss measurements were taken by isolating a particular section of line, allowing various constant flowrates through the section, and observing residual pressures at each end of the pipe section simultaneously, and correcting for elevation differences. There are several sources of potential error in such an experiment. Those which appear to be most significant are the following:

1. Hydraulic transients in the long small diameter lines due to changes in valve settings were minimized by using two-way radios at each end of the test sections. Observers at each station took whatever time was required following valve setting changes to obtain constant pressure and flow readings prior to recording a measurement. The radio communication also insured that the pressure gages at each end and the flow meter quantities were observed simultaneously.

2. Instrumentation errors in flowrate measurements are particularly significant at low flows. This source of error was minimized by carefully checking meter calibration both before and after a series of readings. The flow levels used were obtained by starting with zero flow (to check on elevation differences between increasing flowrate settings including fully open, and then attempting to duplicate these flowrates (and pressures) while closing the valve. If reasonable replication was not obtained, the entire sequence was repeated.

3. Instrumentation errors in pressure measurement are particularly significant in situations with low pressure differentials between each end of the test section (after correcting for

elevation). This source of error was minimized in two ways: First, the test sections were relatively long (4400 to 5400 feet of pipe) thereby producing significant friction losses. Second, careful notes were kept concerning which pressure gages were used at each station, the gage calibrations (throughout the range of pressures encountered) were checked in the laboratory immediately following the field experiment, and field readings were then corrected accordingly.

The three head loss test sections are shown on Figure 13. The 2-½" test section utilized the flow meter and related pressure gage (upstream from the orifice) at the upstream end and a pressure gage just upstream from an existing pressure reducing valve (prv) at the downstream end of the pipe section. The flow was controlled by a gate valve within the prv structure. This means that the non-recoverable head loss through the meter orifice should have been deducted from the total measured head loss. However, this was considered to be negligible since the master meter orifice diameters were approximately ¾ of the pipeline diameters. Whatever error was introduced by failure to make that correction would tend to produce friction coefficient estimates which are on the low side.

The 2 inch test section utilized a temporary pressure gage at a service meter at the upstream end and the flow meter and pressure gage (downstream from the orifice) inside the prv structure at the downstream end (but upstream from the prv). Flow was controlled by a valve inside the prv structure. The effect of pressure loss at the orifice would be the same here as described for the 2-½ inch test.

The 2-½ inch test section utilized the 2 inch meter (even though it was outside the test section), a pressure gage near the 1-½ inch meter at the upstream end, and a pressure gage at an individual meter at the downstream end. This required three way radio communication for simultaneous readings. This section was not subject to the orifice loss error discussed previously for the other two sections.

The detailed friction loss measurements are given in Appendix E and are summarized in Table 11. The results are surprisingly consistent considering the expected error inherent in the pressure and flow measurements. The C value calculations range from 128 to 136 and average 133.

Approximately 2 foot long pipe sections were cut from the three PVC pipe diameters during installation of the master meters. Examination of these sections revealed no apparent change in smoothness of the pipe since construction, and no deposition of sand at any of these locations.

In order to determine what implications should be drawn from the measured friction losses, it is necessary to compare the popular but empirically based Hazen Williams with the much more theoretically sound Darcy-Weisbach equation. Jeppson has converted the Hazen Williams equation into the same form as Darcy-Weisbach and superimposed theoretically accurate values of Hazen Williams C on the relative roughness functions of the Moody diagram (Jeppson, 1976). This analysis demonstrates that the Hazen Williams equation should be used only if the C factor is adjusted appropriately for Reynolds number (and to a lesser extent for diameter).

If the Darcy f factors and Reynolds numbers from Figure 13 are plotted on a Moody diagram they reveal that the entire range of these data represent flows immediately above the hydraulically smooth function. In fact, it is difficult to conceive of any normal design flow condition through small diameter plastic pipe which is not very close to the hydraulically smooth function. In order for the relative roughness function to depart significantly from the hydraulically smooth regime velocities must be greater than 10 ft/sec and diameters must be larger than 6" (Reynolds number greater than 3×10^5). The highest possible velocities through the long lengths of pipe in the Lapoint system were 2 to 4 ft/sec.

The Jeppson (1976) analysis indicates that a Hazen Williams C of 150 would be proper for flows in small diameter PVC pipe only for Reynolds number greater than 10^5. For the flows measured during this study, however, C values of about 140 would appear to be apropriate (Jeppson, 1976, p. 43). The C values suggested by measured head losses at Lapoint were close to, but consistently lower, than 140 (133 mean).

The probable conclusions which should be drawn are: (1) A safe Hazen Williams design coefficient for long lengths of small diameter PVC pipe would appear to be 130. (2) C values as high as 150 should only be used for PVC pipe where flows with Reynolds number greater than 10^5 are anticipated (for 2" diameter pipe this means a velocity of at least 7.2 ft/sec). (3) Because sections removed from these 10 year old lines appeared to be in excellent condition, the apparent decrease in C values over time are more probably due to

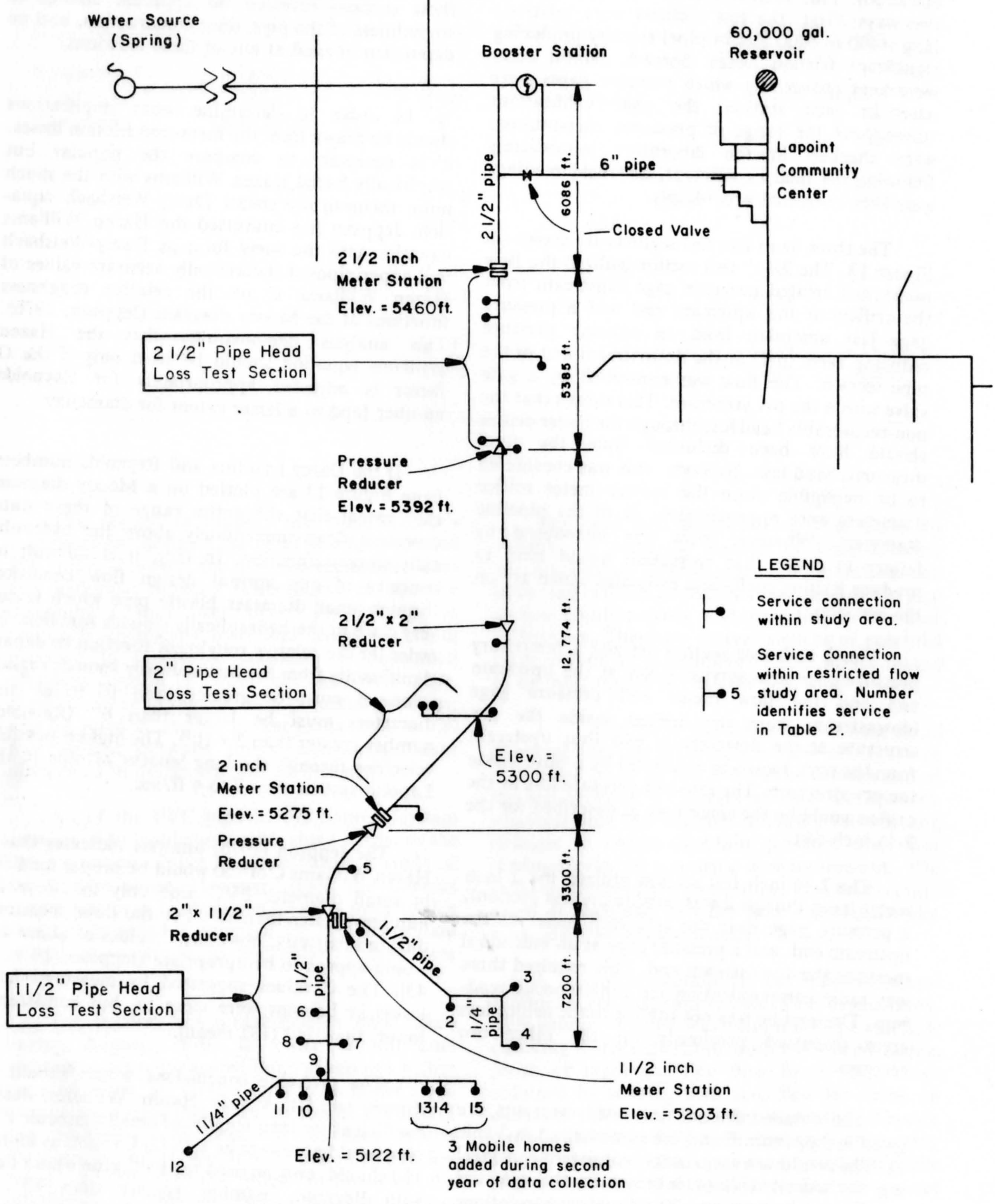

Figure 13. Friction test section locations.

Table 11. Summary of Hazen Williams C coefficients calculated from head loss measurements.

Flow (gpm)	Velocity (ft/sec)	Reynolds Number	Darcy f	Nominal Pipe Diameter 2-1/2"	2"	1-1/2"
12	1.66	14,350	.028			134
14	.811	10,800	.029	136		
16.5	1.46	15,770	.026		127	
16.5	2.28	19,700	.025			135
20.5	1.88	15,850	.0265	128		
24	2.12	22,900	.0245		131	
24.5	3.38	--	--			--
27.5	1.59	21,200	.0248	135		
37	3.27	35,300	.0227		133	
43.2	3.08	41,100	.0213	132		
			Averages	133	130	135
			Overall Average		133	

experimental error (particularly in the original measurements) than to deterioration of pipe capacity over time. (4) Additional field measurements at a site and with instrumentation selected especially for measurement of hydraulic friction loss should be obtained in order to verify the data presented here. Almost no in place verification of the hydraulic smoothness of modern pipe materials is available in the literature.

Comparison with Other Research and Existing Peak Demand Design Standards

In the literature review other publications which include instantaneous demand data were described. The results of these studies are summarized in Figure 14. The labeled data points are maximum measured events from the following studies: Kansas-Allen and Montgomery counties. (Williams, undated); Kansas (Johnson, 1968); Utah (this study), Oklahoma #3 (Goodwin, 1973); Johns-Hopkins East and West U.S. means (Howe and Linaweaver, 1976). The functions labeled "Ginn" were produced from a statistical model based upon measurements in Mississippi (Ginn et al., 1966). All of these studies were related to rural systems except the Johns-Hopkins program.

Figure 14 also includes the Farmers' Home Administration (FmHA) Ohio average and minimum standards as a means of placing the empirical data in perspective. The FmHA national office does not publish recommended national guidelines for design standards but rather allows considerable discretion to individual state offices. This is probably a recognition of climatic and cultural differences within the U.S. that are reflected in water demands. The instantaneous flow standard, however, which appears to be most widely used (by many states besides Ohio) is the Ohio standard. As seen in the figure, there is great latitude between the average and minimum curves. The FmHA national office reports that over 5,000 systems have been constructed according to the minimum curve and that none report any problems meeting demands. As shown in Figure 14, however, the two most detailed studies of instantaneous flows (Lapoint and Oklahoma #3) both recorded demands considerably above the FmHA minimum standard but well below the average standard.

The Kansas measurements are all well below the FmHA minimum standard; so far below the other peaks, in fact, that there is obviously some striking difference either in the nature of the demand or the hydraulic capacity of the systems. The reasons for these differences are not explained by the literature.

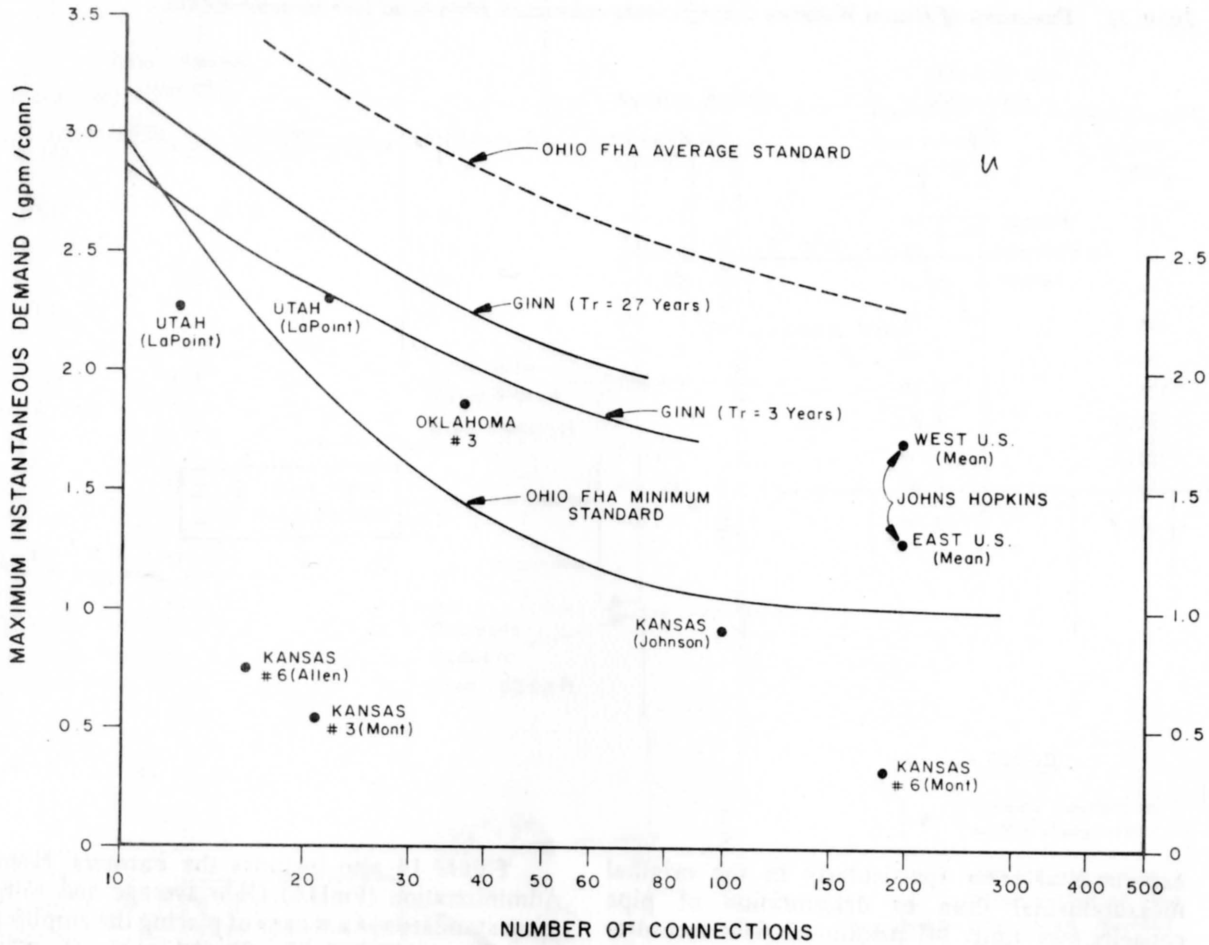

Figure 14. Instantaneous peak flows and FmHA standards.

One gets the impression from several state FmHA offices that although the Ohio minimum standard may be officially allowed, many engineers prefer not to use less than 2.0 gpm unless serious repayment problems would result. A design standard which therefore appears to be widely accepted is a modification of the Ohio minimum which becomes horizontal at 2.0 gpmc for more than 20 services. This would represent a design standard which would fit the Lapoint and Oklahoma (as well as Johns-Hopkins Western) measurements reasonably well except that it appears to be slightly low in the 10 to 40 service range.

Figure 15 includes the same FmHA standards as Figure 14, the Utah State Division of Health standard, demand functions suggested by the Lapoint analysis at two recurrence intervals (3 and 27 years), and the Ginn 27 year recurrence function. The Utah State standard requires capacity well above any of the other standards or measured peaks in the range of interest (over 4 services). All of the statistical models (Utah and Mississippi) and the single point measured peaks (Oklahoma and Kansas) are also below the FmHA average standard but several are well above the minimum standard in important portions of the range. The two statistical models agree surprisingly well, particularly in view of the totally different approaches used in their development. The Ginn model was produced by measuring demand at individual meters and using conditional probability to combine any number of these typical single residence functions to produce combined peak flow estimates. The approach used in this study, in contrast, was to use master meters, thereby directly measuring combined flowrates.

Both the empirical models and the single point demands in Figures 14 and 15 are based upon absolute maximum peak flows, no matter how short the duration. As described in the time duration analysis, the peak events in low density

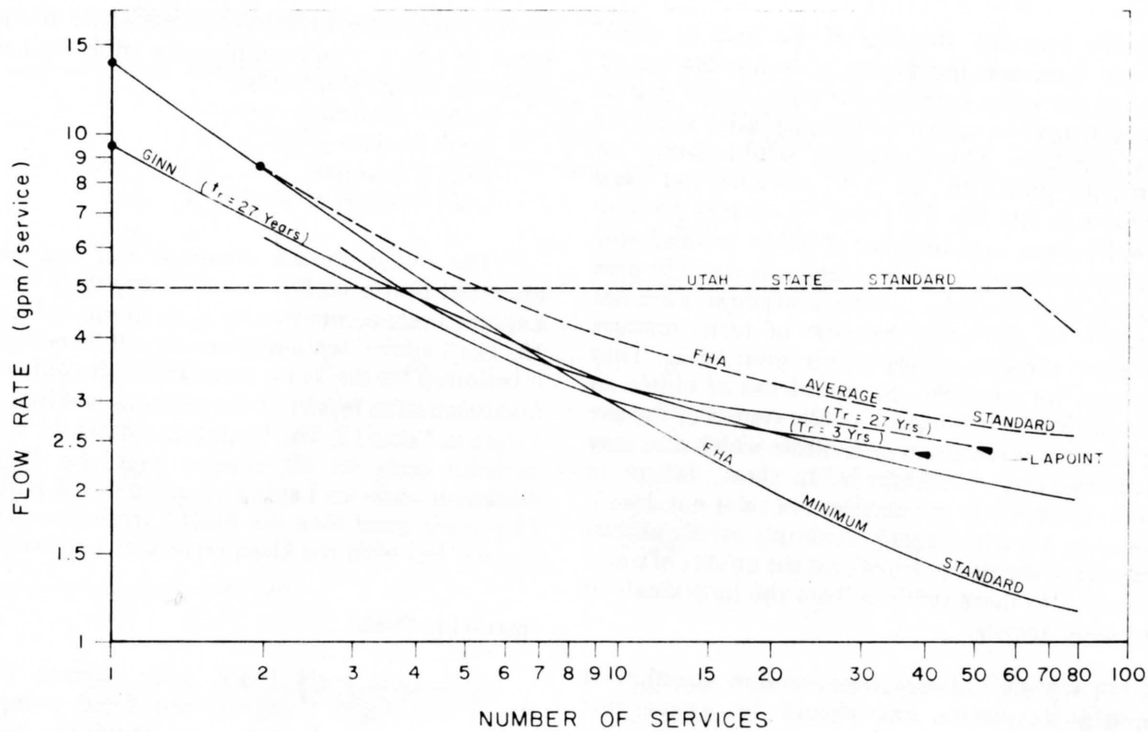

Figure 15. Comparison of measured peak flow recurrence interval (t_r) distributions to existing design standards.

rural systems are extremely short lived, lasting only 1 to 3 minutes. The decision as to what constitutes a reasonable peak event duration for design purposes should involve two types of considerations: customer satisfaction and public health hazards.

If customer satisfaction were the determining factor, surely limiting demand hydraulically once a month for a period of 5 minutes, for example, would be considered as reasonable service. As indicated in the previous frequency and time duration analyses this capability could be provided with significantly smaller pipe than that required by the Lapoint functions shown in Figure 15.

The public health hazard is a more difficult factor to evaluate but probably normally results in higher flow standards than those based upon customer satisfaction. Those public health regulatory agencies which concern themselves with pipe capacities, do so based upon the following rationale: If customer demand exceeds pipe capacity, low pressures in the distribution system will result. This condition will not necessarily result in negative line pressures because the more probable result is a simple reduction of flow from taps generally over the problem area. However, the potential for negative pressures does exist, particularly in rolling terrain or at locations where long adverse pipeline grades occur. In such situations contaminated water could be drawn into the system if there happens to be a situation with no air gap between the system and a source of contamination such as a hose end lying in a puddle or if leaks exist in areas where the pipe is below the water table. The combination of these probabilities (the product of probabilities) existing at the same geographic location at the same time that the peak demand occurs should be the basis upon which design decisions are made.

It would appear that states which allow instantaneous design criteria that are significantly less than 2.0 gpm for small numbers of services (less than 50) must accept the probability of short term periods when demand will be limited hydraulically. In the Utah study a line serving 22 services experienced unit flows over 2.0 gpm for a few minutes a day once in five days. Flows exceeded 1.5 gpm almost once a day. Minimum instantaneous pressures were also monitored during this study and they verified that the flows were not hydraulically limited.

The question of what duration of negative pressure constitutes a health hazard remains. It is, of course, a function of the magnitude of the negative pressure, the size of the leak or other reverse flow, and the degree of contamination (if any) of the water entering the pipe. Public health agencies tend to take a very conservative stand on this question. They naturally would prefer no negative pressures of any duration at any recurrence interval. This may be counter productive, however, even in terms of public health itself. If ultra conservative design standards result in pipe lines so large that the construction projects are not feasible for the small number of rural families involved, the alternatives may be even worse. They include, for example, continued use of either an existing totally inadequate public system or the use of private water sources or facilities which also may represent very real hazards. In short, failure to construct a public system does not solve nor does it not solve a health hazard. It simply avoids official transfer of the responsibility for the quality of water delivered to those families from the individuals to the state agency.

To place the negative pressure question in proper perspective one should be aware that negative pressures occur, of course, almost yearly in some section of most rural (or urban) distribution systems due to pipe ruptures. The mud which enters the line as valves are closed following a line break and during repairs would appear to represent a much more serious public health hazard than infrequent capacity problems where pressures approach zero. It would appear then that acceptance of a design standard such as the Lapoint 3 year recurrence interval function in Figure 15 should be considered conservative even from a public health standpoint for situations where the proposed system demand is similar to that of the Lapoint system (only modest irrigation and stock water components).

IMPACT OF DESIGN CRITERIA ON COSTS

The entire motivation behind development of realistic design standards for rural water systems is the implication that the standards have important effects upon the costs of constructing and/or operating such systems. In order to demonstrate the sensitivity of system costs to variation in design standards, an analysis of both construction and operating costs was included in a previous report (Hughes and Israelsen, 1977). A summary of that analysis follows.

Capital Investment

The fixed cost component of the analysis was limited to pipe cost only. A section of a low density hypothetical water transmission and distribution system was defined for the case study. Pipes were sized by assuming 1977 PVC pipe costs, a friction factor of 130, a minimum pipe size of 2" and the following design standards.

FmHA minimum
Texas FmHA
FmHA Average
Utah Division of Health

The Texas FmHA standard has not been discussed in this report, but it is very close to the Lapoint 3 year recurrence function shown in Figure 13. The Lapoint demand function will therefore be substituted for the Texas standard in the following discussion. The results of the pipe cost analysis are shown in Table 12. The Utah State standard would increase costs by 68 percent over the FmHA minimum while the Lapoint function would require 19 percent more than the FmHA minimum (or 30 percent less than the Utah standard).

Operating Costs

Operating costs based upon current Utah Power and Light Company municipal pumping rates were found to be even more sensitive to design standards than construction costs. The important impact is related to energy demand charges for pumping. Energy cost is an extremely important factor in rural system operating costs because 91 percent of U.S. systems serving less than 2500 population use groundwater as their source (Murray and Reeves, 1972). The impact of this cost component is related mainly to 24 hour peak demand standards, rather than instantaneous peaks, and therefore is not directly related to the design standards of interest in this report. An important exception, however, would be systems or portions of systems which have little or no storage capacity and therefore where pumps are sized to meet short term demands. A summary of both construction and operating (power) costs follows:

Design Standard	Capital Investment (Pipelines)	Operating 50% Use Factor	Costs (Pumping Only) 20% Use Factor
Min.	Reference	Reference	Reference
Lapoint	19% increase	12% increase	42% increase
Utah State	68% increase	38% increase	151% increase

Clearly both capital investment and operating costs are impacted substantially by design standards. The high energy unit cost at low use factors, in particular has important implications for pump design. Consideration should be given to keeping pumps themselves as small as possible in order to almost match capacity with current peak demands (rather than sizing for large increases 20 to 30 years into the future). Pipe lines, however, cannot be replaced so easily and should be sized for future anticipated growth.

Table 12. Impact of design criteria on construction costs.

		DESIGN STANDARD											
		UTAH			FmHA AVG			LAPOINT (3 yr)			FmHA Min		
Pipe	Length	Dia.	Unit Cost	Cost	Dia.	Unit Cost	Cost	Dia.	Unit Cost	Cost	Dia.	Unit Cost	Cost
AB	12,000	8	3.59	48,080	6	2.39	28,680	6	2.39	28,680	6	2.39	28,680
CB	12,000	6	2.39	28,680	6	2.39	28,680	6	2.39	28,680	4	1.45	17,400
CH	11,000	6	2.39	26,290	4	1.45	15,950	3	.99	10,890	3	.99	10,890
HI	14,000	4	1.45	20,300	3	0.99	13,860	3	.99	13,860	2½	.74	10,360
IJ	5,000	2	.55	2,750	2	.55	2,750	2	.55	2,750	2	.55	2,750
IK	5,000	2½	.74	3,700	2½	.74	3,700	2	.55	2,750	2	.55	2,750
Misc. Ends	7,000	2	.55	3,850	2	.55	3,850	2	.55	3,850	2	.55	3,850
Total Cost				128,650			97,470			91,460			76,680
Cost Increase over FmHA min.				68%			27%			19%			Base

PRICE ELASTICITY OF DEMAND

This report has included data on water pricing policy of various systems where available but has not addressed the question of impact of price structure on demand. This should not be interpreted as an implication that rural water demand is not price elastic. In fact, rural water is normally much more expensive than urban and has been demonstrated to be more elastic (or at least is active in a more elastic portion of the demand function) than urban demand. This idea has been addressed in a previous report (Hughes and Israelsen, 1977). However, the price elasticity can normally be demonstrated only for monthly demands since that is the normal interval upon which utilities base their charges. The impact of monthly rate structures upon instantaneous demands is likely very small. Consider, for example, the close agreement between the Oklahoma and Utah peak flows (Figure 14) despite a large difference in unit cost of water ($0.72 compared to $2.00 per 1000 gallons).

SUMMARY AND CONCLUSIONS

GENERAL

Because of the sensitivity of both capital and operating costs to design standards, the rural water planning engineer is in a difficult dilemma. Over design will produce a project that is not economically feasible, while under design will cause periods of low or even negative pressures and, therefore, possible public health hazards. The problem is intensified by the fact that the solution space that the planner is seeking (the region between these two boundary conditions) is much smaller than in the urban setting. The short term demand functions measured at a rural low density Utah system during this study have been analyzed in conjunction with the results of other (very limited) literature on this subject. The information summarized in this report should be helpful in defining the solution space mentioned above.

INSTANTANEOUS DEMAND STANDARDS

The Utah State Division of Health requirement of 5 gpm per residential connection appears to be unnecessarily high, while the FmHA minimum standard does not appear to be high enough for western rural systems with demand functions similar to that of the study area (only modest stock and irrigation demands). The Lapoint 3-year recurrence demand function developed in this report appears to be a reasonable design standard for such systems. Its use implies experiencing some hydraulic limitation on demand once in 3 years for intervals lasting less than 3 minutes (after the design growth allowance has been reached). The demand functions developed during this study represent an intensive analysis of demand at three locations on a single system. A more extensive study which examines variability of demand between rural systems is needed to determine the range of systems for which the Lapoint data are representative. Clearly, a western system in an area which has no supplementary water source for outside irrigation should experience higher peaks than the Lapoint system.

Minimum pressures recorded simulataneously with maximum flowrates established that the measured Lapoint flows represent customer demand rather than hydraulic capacity of the system.

Restriction of Peak Flowrates

Experience during this study suggests that a thin stainless steel orifice inserted in individual service meters is an effective device for physically limiting peak flowrates. The potential flowrate through several Lapoint services were essentially cut in half (12 to 6 gpm average) with almost no resulting impact upon the users' quality of service.

The effect of this restriction upon distribution system peaks was to decrease the maximum flowrate in a line serving 4 families by 7 percent and in a line serving 15 families by 20 percent. The larger line experienced a very significant reduction which could be reduced further by using a slightly smaller orifice.

The orifice concept could be used very effectively either in the design of systems with marginal feasibility or by adding them to a system later as the design growth allowance is exceeded.

Duration of Peak Flowrates

The hydrographs recorded during this study reveal that lines serving as many as 22 rural families experience very random daytime peaks in contrast with the more stable twice daily urban peak events. The higher peaks are also invariably of very short duration (1 to 3 minutes). The peak flow events analyzed in this report are all daily absolute maximums. If a different definition of peak flow event had been defined, say the daily maximum flow lasting 5 minutes, much lower demand curves (9 to 24 percent lower) would have resulted.

Friction Loss Coefficients

Field measurement of friction loss through approximately 1 mile lengths of ten year old PVC pipe at various flowrates were observed. Calculated Hazen Williams C coefficients averaged 133. Previous measurements during the first year of operation produced C's which averaged 151; however, it is not known whether the apparent decrease is due to aging or to inadequate instrumentation during the original measurements. Three sections cut from the pipe lines revealed no observable change in smoothness. Mathematical manipulation of the Hazen Williams equation into the same form as the more reliable Darcy-Weisbach equation indicates that for design flows encountered in small diamter PVC distribution mains a C of 140 should be approximately correct. In view of both the Lapoint field data and the theoretical analysis of the equation it appears that the practice by pipe manufacturers of recommending Hazen Williams C of 150 PVC pipe is definitely not justified.

FUTURE RESEARCH

This study represents an intensive measurement of demands at 3 points on a single rural system. These data represent the only study known to the authors of instantaneous demands on a rural low density system in the arid western United States. Nationally, similar data are available only from single studies in Kansas, Oklahoma, and Mississippi. None of the other studies measured maximum flow and minimum pressures simultaneously thereby preventing determinations as to whether customer demand or hydrualic capacity was being measured.

Obviously a great deal more data both within Utah and other climatic and cultural zones is needed in order to better define the range of this demand parameter. Development of well defined demand functions for other parameters such as 24 hour and monthly peaks will also require extensive research.

The friction factor measurements included as a secondary objective of this study were certainly not comprehensive enough to establish dependable design levels. A research project devoted solely to measuring friction losses in older small diameter PVC and asbestos cement pipe (20 to 30 years of use) would make a valuable contribution to the rural water design field.

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APPENDIX A
LAPOINT CULINARY WATER, INC.
MINIMUM DAILY PRESSURES (PSI)

Date (1975)	No. 1 (1-1/2" pipe)	No. 2 (2" pipe)	No. 3 (2-1/2" pipe)
Aug. 4	52	59	61
" 5	50	58	50
" 6	44	40	30
" 7	46	48	40
" 8	58	64	40
" 9	48	46	60
" 10	58	64	40
" 11	58	80	66
" 12	48	78	62
" 13	48	63	No Rec
" 14	54	60	64
" 15	62	92	36
" 16	60	78	44
" 17	62	90	50
" 18	40	54	46
" 19	56	62	40
" 20	48	80	42
" 21			
" 22	60	80	
" 23	56	80	44
" 24	42	72	60
" 25	56	70	60
" 26	56	74	38
" 27	51	82	36
" 28	48	66	46
" 29	50	96	62
" 30			
" 31	52	90	47
Sep. 1	50	70	46
" 2	53	74	45
" 3	53	74	32
" 4	50	76	38
" 5	60	74	40
" 6			
" 7	50	69	60
" 8	55	74	61
" 9	57	91	60

(Cont.)

Date (1975)	No. 1 (1-1/2" pipe)	No. 2 (2" pipe)	No. 3 (2-1/2" pipe)
Sep. 10	57	84	37
" 11	58	98	36
" 12	59	96	60
" 13	57		
" 14		80	60
" 15	58	84	62
" 16	58	78	25
" 17	58	83	33
" 18	56	76	32
" 19	60	98	46
" 20	58	88	60
" 21	57	88	60
" 22	52	86	44
" 23	50	84	44
" 24	58	88	43
" 25	58	76	44
" 26	59	78	32
" 27	56	75	46
" 28	54	80	40
" 29	51	70	46
" 30	55	84	32
Oct. 1	54	78	46
" 2	48	68	48
" 3	54	96	43
" 4	54	78	44
" 5	56	72	45
" 6	52	80	40
" 7	59	90	39
" 8	58	90	46
" 9	29	84	50
" 10	50	88	44
" 11	57	88	49
" 12	57	88	40
" 13	63	90	41
" 14	76	86	34
" 15	45	80	48
" 16	Gage Frozen	78	48
" 17	"	80	48
" 18	"	80	54
" 19	"		46
" 20	"	80	45
" 21	"	88	46
" 22	"	82	52

APPENDIX B
DAILY MAXIMUM FLOWRATES AND RECURRENCE INTERVALS, SUMMER OF 1975 AND UNRESTRICTED PORTION OF 1976 DATA

Daily peak flows at 2-1/2" meter (22 services).

Date	Metered Flow Q (gpm)	Rank M	M	Flow per Service q_{22}(gpmg)	Recurrence Interval t_r=N+1/M (days)	Probability $P=1/t_r$
Data in Chronological Sequence			Data in Ranked Sequence			
Aug. 8	28	42	1	2.30	43	2.33
9	37	20	2	2.25	21.5	4.65
10	32	33	3	2.23	14.33	6.98
11	33	30	4	2.11	10.75	9.30
12	32	34	5	2.09	8.6	11.63
13	30	40	6	2.09	7.17	13.95
14	29	41	7	2.05	6.14	16.28
15	33	31	8	2.05	5.38	18.60
16	37	21	9	2.02	4.78	20.93
17	38	17	10	2.00	4.3	23.26
18	41	14	11	1.96	3.91	25.58
19	42	12	12	1.91	3.58	27.91
20	32	35	13	1.91	3.31	30.23
21	44	10	14	1.86	3.07	32.56
Sept. 26	43	11	15	1.86	2.87	34.88
27	45	7	16	1.75	2.69	37.21
28	50.5	1	17	1.73	2.53	39.53
29	44.5	9	18	1.73	2.39	41.86
30	46.5	4	19	1.73	2.26	44.19
Oct. 1	45	8	20	1.68	2.15	46.51
2	46	5	21	1.68	2.05	48.84
3	35	26	22	1.68	1.95	51.16
4	35	27	23	1.68	1.87	53.49
5	37	22	24	1.66	1.79	55.81
6	37	23	25	1.62	1.72	58.14
7	33.5	29	26	1.59	1.65	60.47
8	35.5	25	27	1.59	1.59	62.79
9	31	39	28	1.59	1.54	65.12
10	31.5	37	29	1.52	1.48	67.44
11	38	18	30	1.50	1.43	69.77
12	32	36	31	1.50	1.39	72.09
13	38	19	32	1.50	1.34	74.42
14	49.5	2	33	1.46	1.30	76.74
15	41	15	34	1.46	1.26	79.07
16	49	3	35	1.46	1.23	81.40
17	46	6	36	1.46	1.19	83.72
18	33	32	37	1.43	1.16	86.05
19	36.5	24	38	1.43	1.13	88.37
20	35	28	39	1.41	1.10	90.70
21	42	13	40	1.36	1.08	93.02
22	38.5	16	41	1.32	1.05	95.35
23	31.5	38	42	1.27	1.02	97.67

$\bar{q}$ = 1.73
N = 42

Daily peak flows at 2" meter (12 services)

	Data in Chronological Sequence			Data in Ranked Sequence		
Date	Metered Flow Q (gpm)	Rank M	M	Flow per Service q_{12}(gpmg)	Recurrence Interval t_r=N+1/M (days)	Probability $P=1/t_r$
July 7	25	5	1	2.25	18	5.56
10	24	7	2	2.17	9	11.11
16	26	2	3	2.17	6	16.67
Aug. 23	21	15	4	2.17	4.5	22.22
24	18	17	5	2.08	3.6	27.78
25	27	1	6	2.08	3	33.33
26	23	12	7	2.00	2.57	38.89
27	22	14	8	2.00	2.25	44.44
28	26	3	9	2.00	2.00	50.00
29	24	8	10	2.00	1.8	55.56
30	24	9	11	2.00	1.64	61.11
Sept. 1	25	6	12	1.92	1.5	66.67
2	24	10	13	1.92	1.38	72.22
3	24	11	14	1.83	1.29	77.78
4	23	13	15	1.75	1.2	83.33
5	19	16	16	1.58	1.13	88.89
6	26	4	17	1.50	1.06	94.44

$\bar{q}$ = 1.97

N = 17

Daily peak flows at 1-1/2" meter (4 services).

	Data in Chronological Sequence			Data in Ranked Sequence		
Date	Metered Flow Q (gpm)	Rank M	M	Flow per Service q_4(gpmg)	Recurrence Interval t_r=N+1/M (days)	Probability $P=1/t_r$
July 27	16	1	1	4	31	.0323
28	13	10	2	3.750	15.5	.0645
29	15	2	3	3.750	10.33	.0968
30	14.5	4	4	3.625	7.75	.1290
31	12	19	5	3.625	6.2	.1613
Aug. 1	11	24	6	3.625	5.167	.1935
2	14.5	5	7	3.5	4.429	.2258
3	13	11	8	3.5	3.875	.2581
4	11	25	9	3.5	3.444	.2903
5	13	12	10	3.25	3.1	.3226
6	14	7	11	3.25	2.818	.3548
7	13	13	12	3.25	2.583	.3871
Sept. 7	11	26	13	3.25	2.3846	.4194
8	12	20	14	3.25	2.214	.4516
9	14.5	6	15	3.25	2.067	.4838
10	12	21	16	3.25	1.9375	.5161
11	11	27	17	3.25	1.8235	.5484
12	14	8	18	3.25	1.7222	.5806
14	13	14	19	3.25	1.6316	.6129
15	11.5	22	20	3.25	1.55	.6452
16	13	15	21	3.25	1.476	.6774
17	10.5	29	22	2.875	1.4091	.7097
18	12.5	16	23	2.875	1.3478	.7419
19	11.5	23	24	2.750	1.2917	.7742
20	12.5	17	25	2.750	1.24	.8065
21	14	9	26	2.750	1.1923	.8387
22	12.5	18	27	2.750	1.1481	.87096
23	11	28	28	2.750	1.1071	.9032
24	15	3	29	2.625	1.069	.9355
25	10.5	30	30	2.625	1.0333	.9677

$\bar{q}$ = 3.17

N = 30

Unrestricted daily peak flow rated during 1976 at 2 inch meter.

Date	Metered Peak (gpm)	No. of Houses	(M) Rank	(gpmc)
July 12	26.0	13	1	2.08
13	26.2	13	2	2.06
14	25.5	13	3	2.04
15	25.5	13	4	2.04
16	22.5	13	5	2.01
17*	28.5	14	6	2.00
18	20.0	14	7	2.00
19	24.8	14	8	1.98
20	25.5	14	9	1.96
21	26.2	14	10	1.96
22	27.7	14	11	1.87
23	28.8	14	12	1.87
24	29.2	14	13	1.82
25	28.6	14	14	1.82
26	28.0	14	15	1.77
27	26.2	14	16	1.73
28	23.2	14	17	1.66
29	25.5	14	18	1.43

*New connection added
$\overline{q}$ = 1.89
N = 18

APPENDIX C
MAXIMUM DAILY FLOW DURING USE OF FLOW RESTRICTING DEVICES (1976)

Restricted demand at 2 inch meter (15 services)

Data in Chronological Sequence				Data in Ranked Sequence			
Date		Metered Flow Q (gpm)	Rank M	M	Flow per Service q_{15}(gpmg)	Recurrence Interval t_r=N+1/M (days)	Probability P=1/t_r
Aug.	17	24.0	16	1	1.99	17	.0588
	18	28.8	6	2	1.95	8.5	.118
	19	29.3	2	3	1.94	5.67	.176
	20	26.4	14	4	1.93	4.25	.235
	21	27.6	8	5	1.89	3.4	.294
	22	26.9	11	6	1.87	2.88	.353
	23	27.0	10	7	1.85	2.43	.411
	24	27.3	9	8	1.84	2.12	.470
	26	27.8	7	9	1.82	1.89	.529
	27	26.6	12	10	1.80	1.7	.588
	28	28.9	4	11	1.79	1.55	.647
	29	29.1	3	12	1.77	1.42	.706
	31	25.9	15	13	1.77	1.31	.764
Sept.	1	29.9	1	14	1.76	1.21	.823
	2	28.4	5	15	1.76	1.13	.882
	3	26.5	13	16	1.60	1.06	.941

$\bar{q}$ = 1.83
N = 16

Restricted demand at 1 1/2 inch meter (4 services)

Data in Chronological Sequence				Data in Ranked Sequence			
Date		Metered Flow Q (gpm)	Rank M	M	Flow per Service q_4 (gpmc)	Recurrence Interval t_r=N+1/M	Probability $P=1/t_r$
Aug.	3	11.0	14	1	3.95	17	.0435
	4	9.0	21	2	3.87	11.5	.087
	5	10.0	18	3	3.32	7.67	.13
	6	9.0	20	4	3.25	5.75	.174
	7	9.2	19	5	3.25	4.6	.217
Sept.	4	15.5	2	6	3.15	3.8	.261
	5	13.3	3	7	3.12	3.29	.304
	6	15.8	1	8	3.1	2.87	.348
	7	13	4	9	3.0	2.55	.391
	8	12	9	10	3.0	2.3	.435
	9	11.4	12	11	2.92	2.09	.478
	10	12	10	12	2.84	1.92	.522
	11	13	5	13	2.75	1.77	.565
	12	12.5	7	14	2.75	1.64	.609
	13	12.4	8	15	2.75	1.53	.652
	14	8	22	16	2.62	1.44	.696
	17	11	13	17	2.5	1.35	.739
	18	10.9	15	18	2.3	1.28	.783
	19	10	17	19	2.25	1.21	.826
	20	11.7	11	20	2.25	1.15	.869
	21	10.5	16	21	2.0	1.09	.913
	22	12.6	6	22	2.0	1.04	.956

$\bar{q}$ = 2.86
N = 22

APPENDIX D
TIME DURATION CURVES

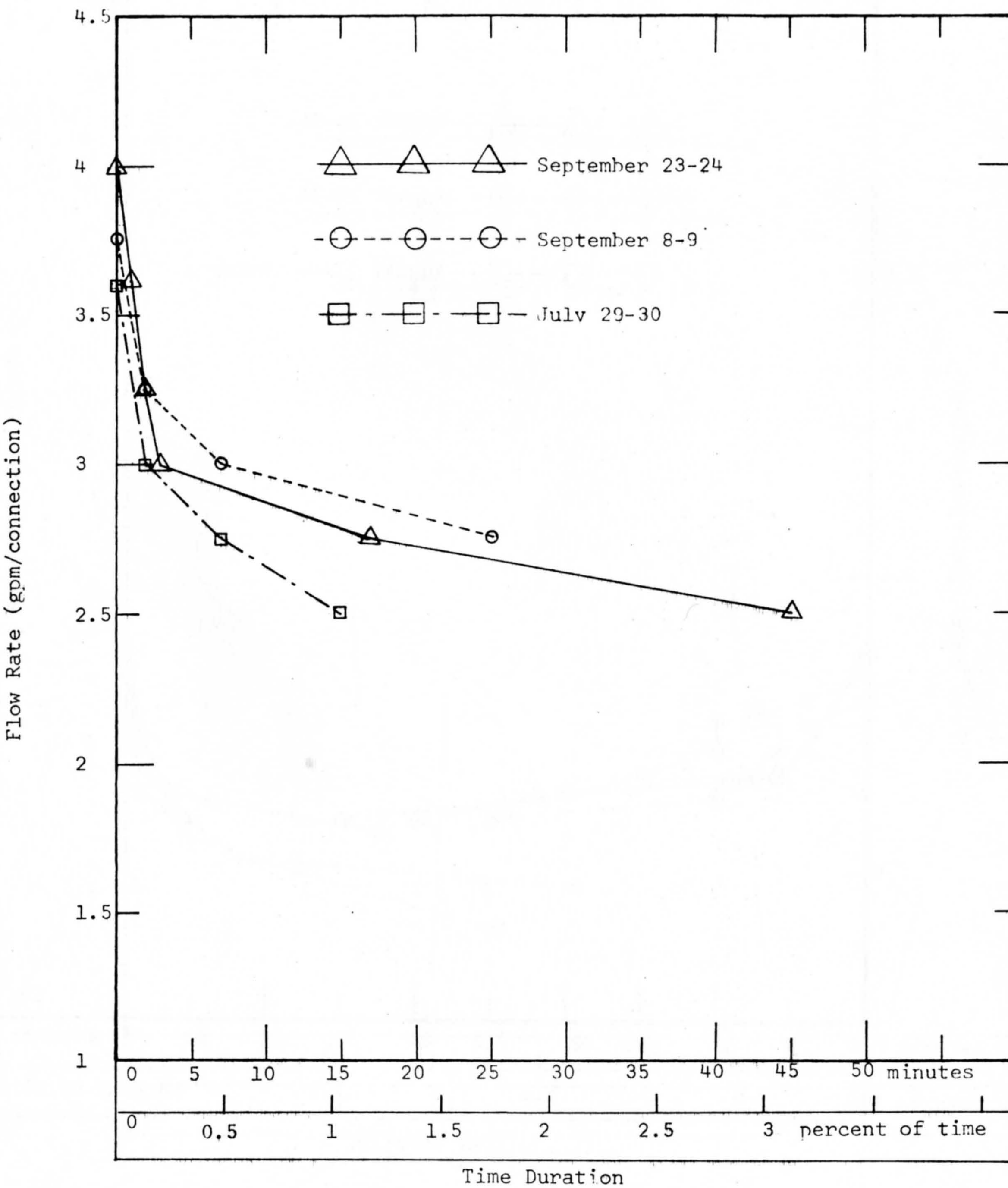


Figure D1. Time duration curves of the top three highest charts for 4 connections.

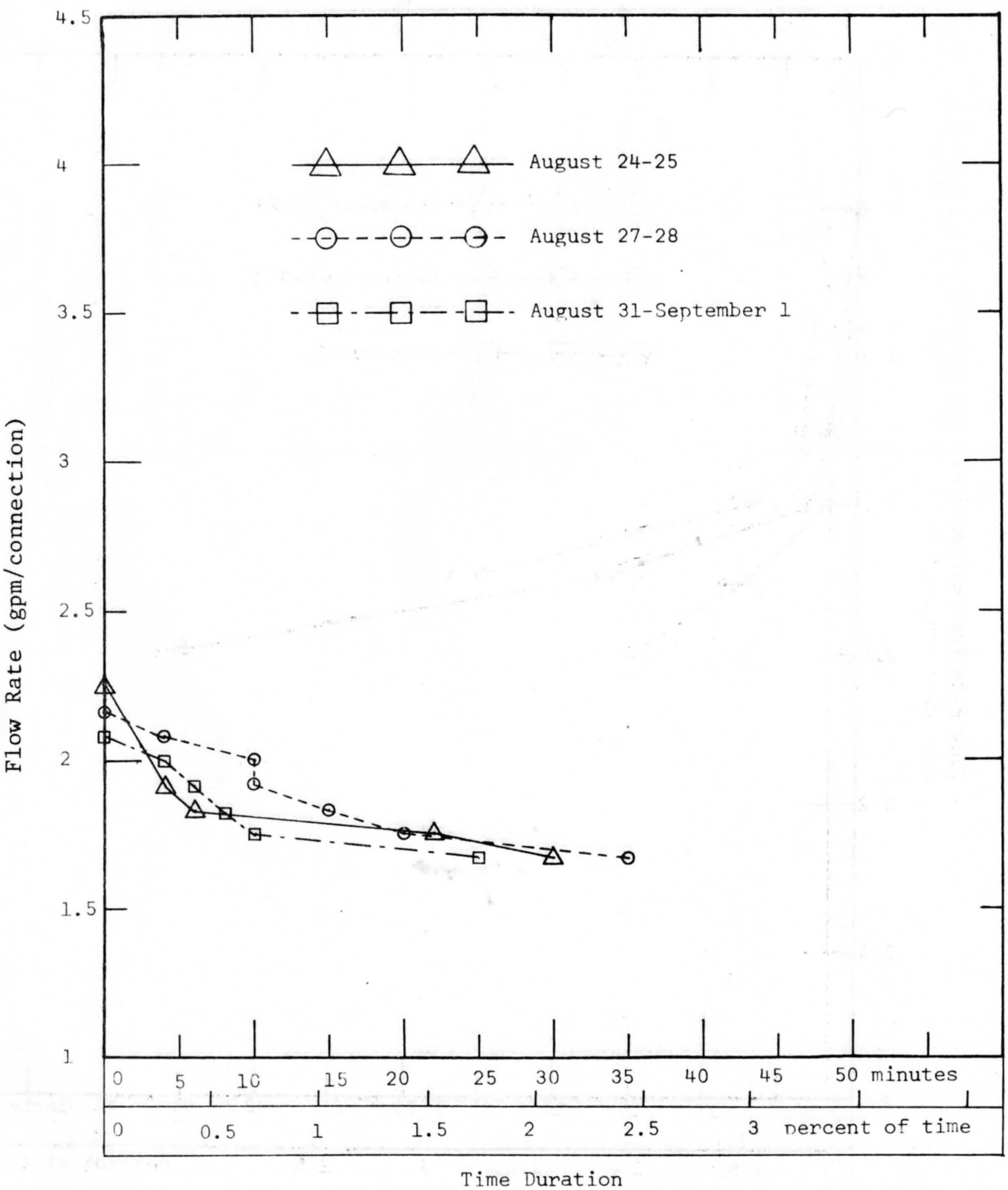


Figure D2. Time duration curves of the top three highest charts for 12 connections.

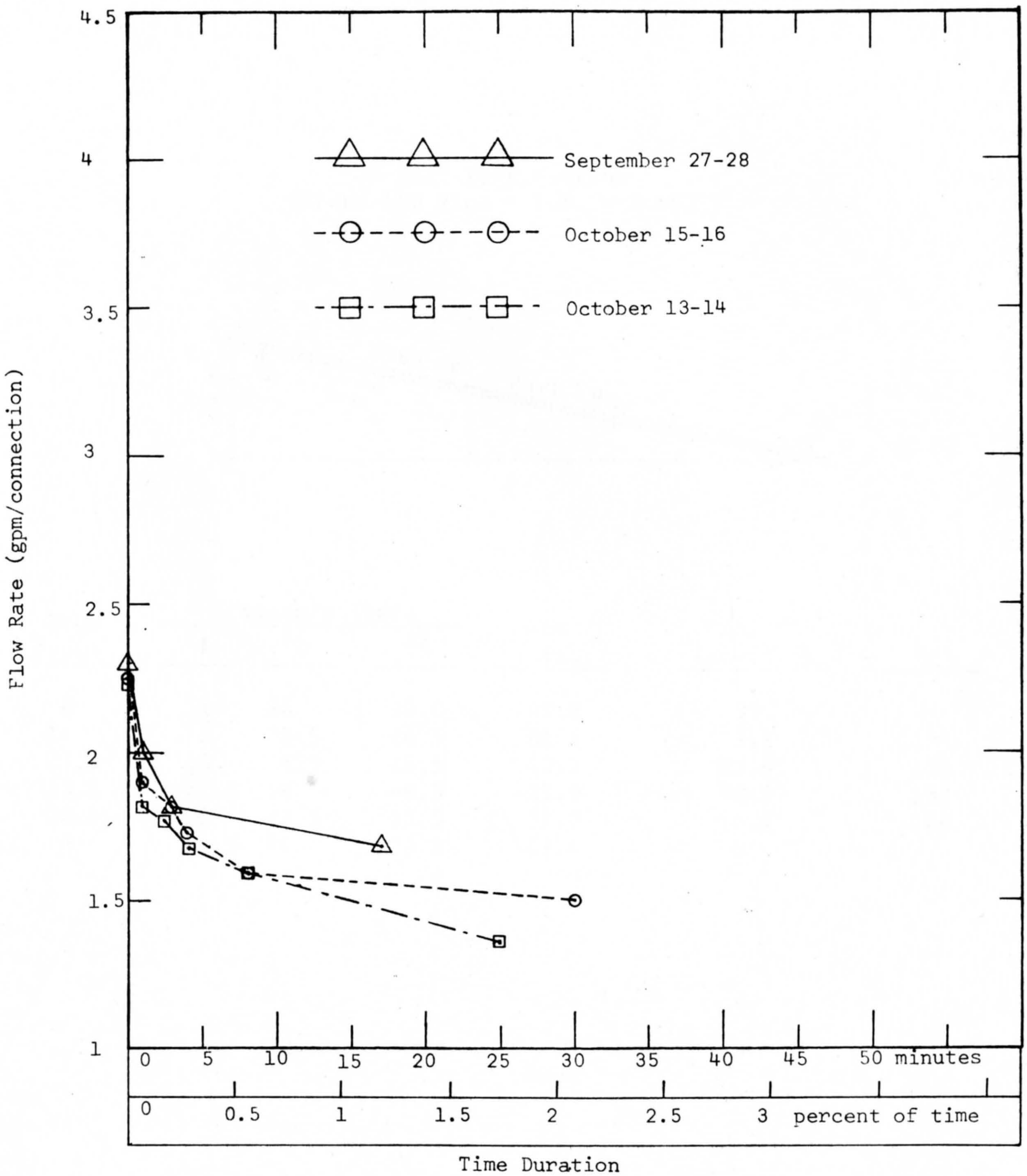
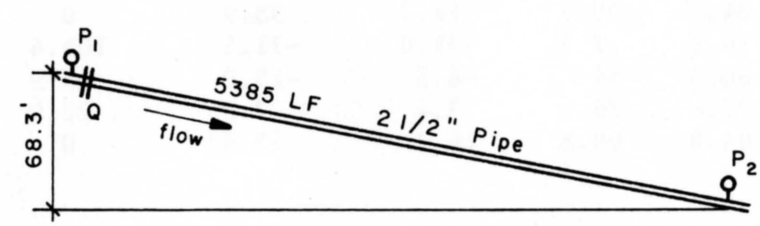


Figure D3. Time duration curves for the top three highest charts for 22 connections.

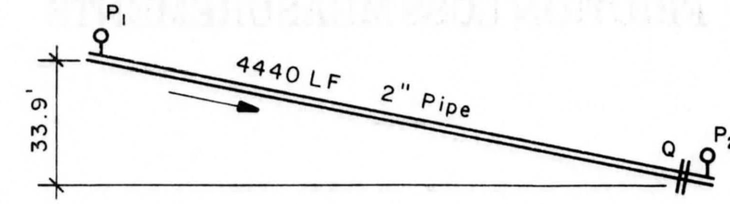
APPENDIX E
FRICTION LOSS MEASUREMENTS

2 1/2" Test Section
(Class 160 Pipe - 1.D. = 2.655")



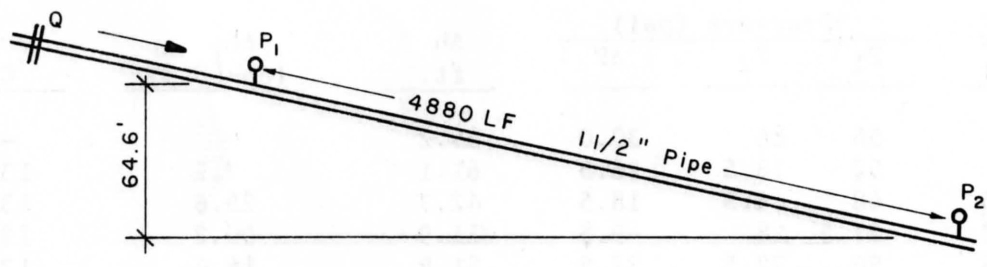
Flow (gpm)	Pressure (psi) P_1	P_2	ΔP	Δh ft.	Δh_f (Δh-68.3)	C
0	56	86	30.0	69.2	0	-
14	52	78.5	26.5	61.1	7.2	136
27.5	48	66.5	18.5	42.7	25.6	135
53.2	27.5	18	-9.5	-21.9	90.2	132
20.5	50	72.5	22.5	51.9	16.4	128
0	55.5	84.7	29.2	67.4	0	-

2" Test Section
(Class 200 Pipe - I.D. = 2.149")



Flow (gpm)	Pressure psi P_1	P_2	ΔP	Δh ft.	Δhf (Δh-33.9)	C
0	84.8	99.5	14.7	33.9	0	-
37	38.5	7.5	-31.0	-71.5	105.4	133
24	60.5	54	-6.5	-15.0	48.9	131
16.5	72.6	76	3.4	-7.85	26.0	127
0	84.8	99.5	14.7	33.92	0	-

1 1/2" Test Section
(Class 200 Pipe - I.D. = 1.72")



Flow (gpm)	Pressure (psi) P_1	P_2	P	Δh ft.	Δhf (Δh-68.3)	C
0	63	91	28	64.6	0	--
16.5	53	48.5	-4.5	-10.39	75	134.6
24.5	27	25	-2.0	-4.61	69.2	208[a]
12	67	76.8	9.8	22.61	42	134.4

[a]Obvious error in gage or meter reading. This value excluded from average C computation.